A HANDBOOK

ON

REINFORCED CONCRETE

FOR ARCHITECTS
ENGINEERS, AND CONTRACTORS

BY

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PREFACE.

In preparing this volume, I have endeavored to produce a reference handbook that would be particularly adapted to the wants of architects, engineers, and contractors. Appreciating the value of a reference book to a designer in any of these branches, and especially when business methods and competition demand an economy of time, such a choice in preference to a text-book, resulted. All due care has been exercised to avoid conflicts with data compiled in the many valuable text-books on the subject.

It was purposed to produce a work treating upon a general form of design rather than upon any one particular or patented system, but to which any of the latter may be applied. The treatment of the many phases entering the design has been carried out along well-known formulæ based upon the theory of elasticity, but modified by the usual assumptions, such as the "conservation of planes" and "Hookes' Law," and not upon empirical formulæ based upon experiments. Attention should be called to the fact that before applying the theory of elasticity to any particular part of the design, a sufficient number of tests were carried out along this basis

to approve it, and determine the coefficients and constants.

The book is divided into four parts: Part I gives a general but concise résumé of the subject from a practical standpoint, bringing out some of the difficulties met with in practice, and suggesting remedies. Under Part II is compiled a series of tests justifying the use of various constants and coefficients in preparing the tables under Part III, as well as bearing out the theory of elasticity. Part III contains a series of tables from which it is hoped the designer may obtain all necessary information to meet the more common cases in practice. It was not intended to cover the more intricate designs, as this is a feature that requires considerable thought and time, both of which may be profitably applied. Part IV treats of the design of trussed roofs from a practical standpoint.

Finally, if this volume will tend to do away with the use of some of the "empirical formulæ" and "rule of thumb" methods of designing reinforced concrete structures, and tend to concentrate all toward a standard and universal system, as well as remove some of the prejudicial influences at work tending to demerit its worth because of unfamiliarity with its design, it will have accomplished its purpose in the mind of the writer.

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PART I.

TENSILE STRENGTH OF CEMENT.

HANDBOOK ON

REINFORCED CONCRETE.

THE tensile strength of a cement enters into the design of a reinforced concrete structure only indirectly. However indirectly as it may be, it is of the utmost importance, since the possibility of realizing a satisfactory design depends entirely upon the obtaining of a satisfactory value for the same. The prime object of knowing the tensile strength of a given cement which is being used on works of magnitude, is to safeguard the owners that they are receiving from the makers the quality of cement specified by the architects. Thus it may be seen that the time when this factor enters the problem is not during the design, but during the erection of the plant.

SPECIFICATIONS. — For instance, a cement is sometimes specified to be calcined from given proportions of given constituents, which are known to render a first-class cement of a high tensile strength. This measure, although used for caution, seems too exacting upon the cement makers, as doubtless there are secrets in the manufacture of the cement, known only to the makers, which are of great value to it, and which would be entirely upset by such stringent requirements. It

should rather be specified, and more properly, that the cement delivered on the works must stand a certain tensile strength per square inch when made into the standard briquette of neat cement, and allowed to set in air for a given time before testing, or shall stand another stated stress per square inch when made as before, but allowed to take its initial set in air, and then immersed, and allowed to remain a stated time in water of a given temperature before testing. Sometimes one or the other of these figures of tensile strength per square inch is given, and the second given as a ratio in terms of the first that must not vary over a certain amount. In either case, the briquette should take its initial set without a perceptible rise in temperature.

Again, it may be specified that the cement in question shall be thoroughly burned during calcination. To satisfy ourselves in this respect, we have but to carefully watch the above-mentioned briquettes or other samples while setting. Should these show a rise in temperature while setting, we are at once convinced that the cement was not properly burned. Yet, if this fact be lost sight of at this time, the results from the tensile strength will show that something is wrong, and if this something reduces the test below the fixed specified amount, we are justified in condemning the lot, provided sufficient tests to give an average of the lot show the deficiency, without troubling ourselves as to the exact cause.

Thirdly, it may be specified that the cement be ground to a certain degree of fineness by requiring a certain part, or all, to pass a sieve of specified number, and all, or but a small per cent, to be retained on a second sieve of smaller mesh. If the cement be improperly or rather too coarsely ground, there will be grains in more or less numbers of too large proportions to bond with the finer and more uniform mass, and the only mission these can have is to act as so much sand, and necessarily lower the tensile strength. So, again we resort, or should resort, to the results of the tensile tests. But this fault may also be detected by weighing, for it is generally known that, bulk for bulk, a coarser ground grade will weigh in excess over a finer ground grade. It is not practical to be too severe with this measure, for there is danger, if the brand be too finely ground, and especially if the sand used be fairly coarse, that the particles of cement will be too small to fill the voids in the sand, unless we make ourselves doubly sure, and use a larger ratio of cement to overcome the danger, which, to say the least, is a remedy too extravagant for the most scrupulous.

Fourthly, caused by the improper mixing of materials, or by insufficient burning during calcination, a pat of neat cement when worked up in the hand, and placed upon a piece of glass under water, will creep and draw up along its outside perimeter where it meets the glass. Also, primarily due to the same cause, a pat of neat cement

made as before, and placed under water, will blow, liberating bubbles of air and indicating chemical action taking place. Both these influences tend to lower the tensile strength.

Attention has already been called, first to the mixture of constituents composing the cement; second, to the proper burning of these constituents; and thirdly, to the degree of grinding after the calcining process. These I consider the three important steps in the making of cement, and a slip in any of these during manufacture renders a product totally unfit for use. To guard against the use of any lot, deficient in this manner, and especially in reinforced concrete construction, is the prime object of what has been written; and finally, as a safeguard, I make an urgent appeal for the more general adoption of making tensile tests, and to an extent to give a fair average of the lot in order to show up the quality of the cement which is being used at any time upon the works.

Now, the brand of cement which has been specified, and which is supposed to meet all the requirements, has arrived on the works. To be sure, one has his faculties of sight and feeling, which can be used to good advantage in passing superficial judgment upon the lot, provided his judgment has had the necessary training through experience. For instance, the color of the lot will tell the more experienced in a general way the composition of the mixture, when the locality from

which the constituents were taken is known, since the ingredients vary greatly in texture in different localities. This, at the best, can be of only passing importance, as the exact proportions of the constituents, which so largely affect the chemical reaction to make the proper compound, will not always appeal to the eye in the same way when so many things enter into the process which may upset any fast rules. But very fortunately there is the privilege of applying the tensile tests to satisfy oneself that the combination of ingredients is such as not to impair the strength.

Again, the sense of feeling may be used. The expert can, by running his hand through the lot and by rubbing together the particles, in a measure tell whether there are too many unground or too coarsely ground particles to affect the results so as not to meet the requirements. Once more, if one is not thus skilled, and in all cases as a precautionary measure, the tensile strength should be relied upon to determine whether the lot is not too diluted by coarse particles, before accepting the lot. Thus, one has his faculties to guard him against two of the many faults; and how often does not this suffice to accept a lot, which, had tests been taken, would never have been unloaded from the car.

In summing up, it may be expected that two lots of cement, made from the same ingredients, taken from the same locality, and mixed in the same proportions, will, if made into neat cement

briquettes, using the same care as regards mixing, proportion and temperature of water used, and the place and conditions of setting, give fairly uniform results by tensile tests. Hence, it remains for the architect merely to fix upon the tensile strength of a cement known to be good when allowed to set for a given time under given conditions, and to see to it that every shipment, when a sufficient number of samples have been taken to warrant the average lot, shall meet the requirements, when the briquettes have been made, set, and tested under similar conditions which governed the standard. No little stress should be laid upon this matter, for it seems to the writer, that in works of magnitude, where every other precaution is taken to insure the obtaining of proper materials, and where a large amount of money is at stake, this primary function of quality and endurance to the structure, second not even to workmanship of mixing and depositing the concrete, should not be overlooked nor deemed too expensive to maintain throughout the time during which the concrete is being placed. By what has just been said, do not imagine that the workmanship is a matter of inconsiderable importance. On the contrary, it is second in importance only to the grade of cement. It should be remembered that the life and endurance of the structure are dependent upon both conditions, and anything lacking in one cannot be compensated for in the other, and the whole suffers in accordance.

Classification of Trap Rock Sizes.

It seems to the writer that one of the necessary steps in the specifications of the architect is to classify the sizes of trap rock that may properly be used in the different parts of the building. Thus, it may be arbitrarily fixed that the sizes of rock to be used in the foundation must all pass a $2\frac{1}{2}$-inch ring, and all be retained on a 1-inch screen. For exterior walls and piers, where the sizes of the same will permit proper rodding, it may be specified that all rock shall pass a 1-inch and all be retained on a $\frac{1}{2}$-inch screen. For girders, beams, and floors above the steel members, also for columns, all rock should pass a screen of $\frac{3}{4}$-inch mesh, and all be retained on a $\frac{1}{4}$-inch mesh. Below and around the steel members in girders, beams, and floors, in order to obtain proper rodding and perfect work, no stone should be used that will not pass a screen of $\frac{1}{4}$-inch mesh, and should include everything under except the dust. This is ordinarily known as pea size.

Caring for Crushed Stone upon the Works.

Under the heading of Classification it may be perceived that in cases where a crusher is used on the works, all grades of the trap rock may be used from $2\frac{1}{2}$ inch down, excluding the dust. As screened, the different grades should be deposited in bins or piles properly labeled, so there can be

no mistake made by the man in charge of the mixer or the different gangs of men when there comes a call for a change in mixture.

If, on the other hand, the trap rock is received on the works by the carload, it should be seen to that each carload is labeled as to its grade, and that it is unloaded into its proper bin or pile. As a safeguard where the rock is received by the carload, and not inspected as to its grade, and especially when coming from an unreliable yard, it is well to run the rock through screens so arranged and spouted beneath that the different grades reach their proper piles.

Accordingly, four bins or piles will be required; and if circumstances will allow, and machine mixing be adopted, the hopper feeding the mixer should have four compartments. Yet this is not absolutely necessary, for it is not probable that there would be calls at any one time for the four grades from any one mixer. Before charging the hopper, or its different compartments, it is absolutely necessary that the rock be thoroughly washed free from dirt and dust in order that every opportunity may be given to the cement to completely coat the exterior surface.

No little stress should be laid upon this matter of classifying the different grades of rock, and keeping each within its sphere for its proper installation in the building. For each grade of stone there is a definite amount of sand and a fixed proportion of cement within limits required

to make a homogeneous concrete; and without this homogeneity of mass, we are putting weak links in the chain, just as would be done provided we allowed poor cement to enter. As there is a safeguard against this latter, so there is against allowing the different grades to be interchanged, and this is care. To be more explicit in regard to the consequences which are bound to exist without this due amount of care, let me add: Supposing the mixture which is being run through the mixer is for beams or floors, and the measuring devices are set for the proper amount of sand and cement for a $\frac{3}{4}$–$\frac{1}{4}$ gauge stone. Through carelessness, let us suppose that the mixer hopper has been charged with a few buckets of the $2\frac{1}{2}$–1 gauge stone along with the $\frac{3}{4}$–$\frac{1}{4}$ gauge. When this enters the measuring hopper, there is neither time nor ready means of changing the sand and cement to agree, provided the tender knew enough. Consequently, the larger stone goes through with the proportions of sand and cement for the smaller stone, — in other words, with voids unfilled, and the concrete far from being homogeneous. But the trouble does not end here. The mass, already diluted, is deposited upon the floor, and shoveled into the beams, and other men follow along with tamps to rod the same into place. Although going through their usual mechanical motions, what is the result? With the large size of stone it is impossible to work them into a small beam, or around steel members without allowing voids to

form, and again we are sacrificing homogeneity. Thus a slip in one respect has weakened the chain twofold.

Sand.

Where practicable, two grades of sand should be specified, — one to be a very coarse and angular crushed quartz; the other to be a finer river or bank sand, also angular. The proportions of the two may be determined thus: Take a given bulk of the coarse sand, and determine the voids after the manner described later on. The ratio of the voids to the original volume of coarse sand will be the ratio of fine sand to coarse to be used. The two grades of sand should be measured out, thoroughly mixed in the above proportions, washed free from dirt, and deposited in a pile or bin ready for the sand compartment of the mixer.

If but one grade of sand can be conveniently obtained, it should be of medium grade, very irregular as to size of granules, and, of course, sharp and angular. Besides, it should be tested at frequent intervals for voids to insure the proper amount of cement at all times to obtain a homogeneous mass.

Proportions of Ingredients for the Various Mixes.

Now that we have the ingredients for making the concrete, — namely, the cement, which has

passed the tensile requirements, the cleaned sand in its compartment, and the rock in its various compartments fixed by the arbitrary grades established, — next comes the fixing of the ratio of these ingredients, determined by the grade of rock and the grade of sand, to make a proper concrete as regards homogeneity.

To do this, take a form impervious to water, that will hold just one cubic yard by its actual inside dimensions. Fill this shovel by shovel with the grade of stone from which is required the proper mix, compacting the same as much as possible. Then obtain, by measuring the volume of water required to fill the voids between the stones, the amount and ratio of the voids to the whole. Then remove the stone and water, and with the volume of sand as just determined by the volume of water thoroughly mixed with the measured cubic yard of stone, replace the mixture into a water-tight form of the same width and length, but of somewhat greater depth than before, tamping the same well as it is placed shovel by shovel into the form. Then the increase in depth will give the relative increase in proportions by adding the sand. After leveling and tamping the contents into the form, measure again the amount of water required to fill the remaining voids just to the level of the top of the sand and stone. Remove, and with other stone of the same grade, and other sand of like grade, and both of the same volumes already determined, and with the

volume of cement determined by the last measurement of water added, all thoroughly mixed, replace the same into the measuring form shovel by shovel, tamping and leveling as before. Again note the increase of height and hence the increase of volume by adding the cement. Undoubtedly we could add a considerable volume of water to the mass before same stood at the level of the mixture in the form, showing voids remaining unfilled. These, however, are due to improper mixing of the materials, and insufficient ramming into place, and should be compensated for as stated farther on. Now we have determined not only the proper ratio of ingredients for the grade of stone and sand in question to make a homogeneous mix, but also the ratio of the final to the initial volume, which will be from 1.1 to 1.3, depending upon the grade of stone. So we may expect that every cubic yard of crushed stone will fill a space in the structure of 1.1 to 1.3 cubic yards after the same has been made into its proper mix of concrete.

After water has been added, which should be enough to make a plastic mass in order to insure the filling of the mold properly, and especially plastic where it is difficult to incorporate the mass thoroughly by rodding, tamping, and rolling, the mass will have gained in bulk nearly in proportion to the water added, provided that the voids have been properly filled when mixed dry, that the cement is in proper condition as already deter-

mined by the tensile tests, and that the rock and sand have been thoroughly wetted beforehand, as both have a certain avidity for water depending upon climatic conditions. This gain in mass will probably amount to 2 to 5 per cent over its volume when dry, depending upon the amount of water added. While setting, this water is gradually absorbed by chemical action and outside influences, and the mass gradually diminished, tending to assume its normal state when dry. Thus, what is generally known as the shrinkage of concrete during set is merely the tendency of the mass to attain its original bulk when dry.

As previously mentioned, voids remain in the mass after seemingly the proper proportions of ingredients have been fixed. These are bound to occur, for in practice, because of so many variables entering to upset the best of calculations, it is impossible to obtain nearly the results which are obtained when determining the proportions. Then again, when the plastic mass is deposited into the molds, it is impossible to prevent a leakage of water, and this leakage will carry away with itself some of the cement. To overcome such difficulties, which must necessarily happen, we have to resort to a factor of safety, as we might term it, by adding to the proportions already determined an excess of cement varying from 5 to 10 per cent as best seems required to meet each particular case.

In what has been said, it may appear how

difficult it is to obtain the proper proportions of ingredients, even when the utmost care is taken. Hence, all the more reason for being careful.

In summing up the matter of proportions of ingredients, we may derive some general figures, which we will term mixes, all of which can be only approximate, and all subject to the variables already mentioned tending to change them more or less. For instance, for the 2½-inch–1-inch grade of rock, the proportions of cement to sand and to stone should be about 1–2½–5. This we will term a 1–2½–5 mix. For the 1-inch–½-inch grade of work, a 1–2–4 mix seems to be about the proper thing in round numbers. Likewise from the ¾-inch–¼-inch grade, a 1–1½–3 mix results, and from the pea-size grade, a 1–1–2 mix results.

By comparing these mixes with what has been said concerning the proper sizes of rock for individual locations in the structure, it can be readily seen that

Foundations require the	1–2½–5 mix
Outside walls, piers, etc., require the	1–2 –4 mix
Girders, beams, floors, and roofs above the steel members, also inside columns, require the .	1–1½–3 mix
Girders, beams, floors, and roofs below the steel require the	1–1 –2 mix
Wearing surface for floors requires	1 cement–2 sand

Incorporation.

After the different mixes have been settled upon, and we are sure these are coming from the mixer in due form, the next important step seems

to be the distribution of these mixes into their proper places. Whatever else goes wrong, it is necessary to have the 1–1–2 mix to work around the steel in the beams and girders and for the first spreading upon the floor to receive the steel. If not, the penalty is paid later on, when the false work is stripped, and a honeycombed surface presents itself. There is no excuse for the 1–2½–5 mix ever reaching the superstructure, so we may eliminate this from our cares. It should be carefully watched that the 1–1½–3 and the 1–2–4 mixes reach their destination, but the danger resulting from the interchanging of these two mixes is the least of any of the combinations.

That proper incorporation should result by careful and scientific rodding, cutting, tamping, and rolling, the plastic mass must be a realized fact, and this can be so only by using all care and by putting the proper man in the proper place. But first of all, before we can rod, tamp, or roll the mass successfully, we must have a mass that is in such a physical state as can be so rodded, tamped, or rolled. By this is meant a concrete made from a moderately slow-setting cement, one containing the proper mix, and one sufficiently plastic. A cement that will begin to take on its initial set to any extent within half an hour after mixing has no place in the superstructure of a building, and yet how often do we see such a cement in use that will set up so hard as to require hoeing or picking out of the bucket.

How can this be properly rodded, tamped, or rolled? The time of thirty minutes' grace, as we may term it, is arbitrarily fixed upon as under ordinary conditions; the rodding, tamping, rolling, leveling, and walking over a section require that amount of time at least, before it can be left for nature to take her course.

In many cases it is required that the top finish shall be floated on immediately after the base is laid. This seems to the writer quite unscientific and very impracticable. It is held by the supporters of this method that the strength of the floor is increased, and that perfect bond between the base and the finish is realized. That this latter is accomplished, no one can dispute. In regard to the former supposition, there seems to be just grounds to take exceptions. If any strength is gained thereby, it is by adding more area to the concrete to resist compression. Already, before the addition of the 1-inch finish, the floor is as able to resist compression as it is tension. Any addition of strength to one link of a chain does not increase the durability of the whole chain. It may be argued that this will increase the moment of resistance of the steel, which undoubtedly would be true if it could be practically realized, but this is not the weak point of the floor if properly designed. The weak point is the adhesion of the concrete surrounding the steel members. By adding the finish at this crucial time, it would be necessary to keep walking over the

soft base, which has already begun to set; and in so doing, you are all the while destroying the bond between the setting concrete and the steel, thus impairing the adhesion between the two, without which, the steel is more or less unsupported, and, when undergoing tension by bending, may be so separated from the concrete that the latter is unable to take the stress, caused by the elongation of the steel fibers while bending, from the steel. In other words, instead of having a compact mass so constrained, one member by another, that any stress in one may be transmitted to another, and the entire stress distributed throughout the number of members, we have a more or less mutilated and disconnected whole. All our former carefulness to obtain a homogeneous compact mass has been set at naught, and we are only undoing what we have already tried so hard to do. Then there is another consideration. The top finish should be considered as the top floor of a house or mill, a part which takes the wear. What strength it imparts to the lower floor, we do not stop to figure or think of. It is put there for one sole purpose, to take wear, no more nor less. To take wear, the cement finish should be uniform throughout, both as regards breadth and depth, and to attain this, the top surface must be screeded off perfectly level. To do this, the screeds cannot be set on a soft surface and the same leveled both ways over any considerable amount of area with the ordinary level

and straight edge. It requires the accurate work of an engineer with an instrument to establish a level grade by setting nails in the base floor after the latter has set for twelve to twenty-four hours, and for careful workmen to bring the screeds to the nails, to attain anything like proper results. Provided the mason could level the screeds properly, how long would they remain so, resting as they would on a soft base, which is all the while being disturbed by walking over it? The effect of screeding off a floor, out of level, is for the water to flow by gravity to the low places, carrying with it cement from the high places. The cement which has collected in the hollows, after the surface has been floated and finished, forms into a thin skin with no body, and instead of bonding with the particles below, seems to keep apart from them, sets slower, and later on when the floor is put in use, scales off by wear. The high places have lost a considerable amount of the cement which should be there in order to float and trowel the same to a smooth, hard surface, but instead, the surface is rough and sandy, in such condition that trucking and wear keep removing the rough particles; hence the spot or spots grow more and more uneven, and more extensive.

Again, the water insuring a proper set, has run from the high places to the low places, allowing the latter to become oversupplied. Consequently, the surface is not uniform, and when one part is

ready for floating or smoothing, another part is not ready, thereby increasing the chances of allowing one part to go too long to be properly worked. Ultimately, the high places set faster than they should, because of insufficient water, while for the opposite reason, the low places do not set so quickly. The only thing that can be expected, under such conditions, is a network of cracks between the two surfaces, and these expectations are usually realized. Now what has been gained? Strength may have been added to the floor by hurrying up the finish, but instead of obtaining a surface to meet wear, a rough substitute is obtained, uneven in surface, and far from homogeneous, and the consequences, a series of repairs to keep the floor intact.

On the other hand, if the base is allowed to set for twelve to twenty-four hours, all danger of destroying the adhesion between the concrete and steel has passed. The surface of the base is uneven, it may be scratched with a rake to make it more uneven, and it should be well-wetted down before applying the finish. With these things attained, there is no reason why there should not be a sufficient bond between the base and the finish. Then everything is favorable for obtaining a level surface by bringing the screeds to nails set in the base to exact grade, and the men applying the finish may have daylight by which to see what they are doing, all of which tend toward good results.

Protecting Newly-laid Work.

Now, having obtained a floor surface properly laid to take wear, the next step in sequence seems to be to properly care for the results already obtained by protecting them from outside influences. We have seen that, if some portions of the surface set earlier than others, the result is a network of hair cracks dividing such surfaces. It is perfectly well-known that concrete of a given mix requires a given amount of water to cause proper setting; any water in excess will be left free to be absorbed by the air; any deficiency will leave parts of the concrete improperly set. Along this line of reasoning, it may be easily seen that if a newly-laid floor surface be left unprotected from the sun on a dry, hot day, the result will be that the rays of the sun, and the surrounding dry air will extract from the upper surface a considerable amount of water. Being robbed of what it should require to set properly, and at the same time, being forced in places to set much earlier than it otherwise would, it tends, by shrinkage of the affected portion, to separate the same from the lower and less-affected layers. The result is imperfect bond between the successive layers and cracks between the more or less effected parts of the exposed surface, which, of course, are detrimental to wear, as well as unsightly in appearance.

To overcome such influences, and to hold nature somewhat in check, the writer would suggest that

just as soon as an area, however small, had received its final troweling, and had become consistent enough to support a covering of burlap or canvas, the same be spread over the area and wetted by spraying gently with a hose. As the surface below becomes harder and harder, water should be sprayed over the covering in increasing amounts, and just as soon as the surface will allow it, the whole covered area should be flooded. In about twenty-four to thirty-six hours, this temporary covering may be removed, when the surface should be covered with a layer of sawdust or fine sand sufficiently thick and evenly distributed to protect all parts. Preferably sawdust should be used, as the particles are more elastic, and are not so harsh upon the finished surface when walked upon. Coarse sand or gravel should be infrequently resorted to, for upon areas which require much walking over, the surface will become very spotted and marred, because of the coarse particles being pressed into the surface not already hard. This covering should be flooded with water, and kept wet, or at least damp, just as long as practicable, or until ready for occupancy, if possible. This covering not only protects the surface from atmospheric influences, but also from wear which the newly-made surface could not otherwise withstand. In this way, work upon or above the newly-placed area may not be delayed more than thirty-six to forty-eight hours.

PART II.

TENSILE STRENGTH OF CONCRETE-STEEL.

TENSILE STRENGTH OF CONCRETE STEEL, OR THE EFFECT OF STEEL MEMBERS UPON CONCRETE WHEN EMBEDDED IN THE LATTER, AND THE WHOLE IS UNDERGOING TENSION CAUSED BY BENDING.

When we come to determine the tensile strength of concrete, when lying just around or between steel rods, as in case of the layer containing the tension members near the bottom of a concrete-steel beam undergoing tension caused by bending, we have a more complicated problem to deal with than with the simple tensile strength of a cement briquette. Experiments galore have shown just how much may be expected from a tensile specimen of concrete alone, when the history of its manufacture is known, and the treatment it has undergone, and under what conditions. It yet remains for experimenters, after many and careful trials, to enlighten us concerning the influence the steel members have upon the concrete when embedded in it, and the combination is undergoing tension caused by bending. That this influence is enormous, no one can dispute when examples, and these from reliable sources no

doubt, are cited, where tests to destruction of concrete-steel beams, by bending, have ruptured the steel members. Since the ratio of the concrete section to the steel section in the layer containing the steel has not been furnished us, we are unable to compute the stress in the concrete section, which, indeed, is to be exceedingly regretted. We are, however, quite justified in saying that in such layers, where, for instance, the section of the concrete between the steel members just equals that of the steel itself, that the stress in the concrete is one-tenth that in the steel, allowing the ratio of the modulus of elasticity of the concrete to steel to be 1 to 10.

As there are several cases in actual practice where the equality of the sections of steel and concrete in the layer in question actually exists, and where, from the actual loading, the stress per square inch of steel figures 15,000, it is but fair to assume that the stress caused in the concrete thereby is one-tenth of this, namely 1,500 pounds per square inch.

Critics who are wont to call the ultimate tensile strength of concrete 200 to 300 pounds per square inch will no doubt either laugh this reasoning to scorn, or will refuse to use a construction which imposes such exacting conditions upon a material weak in itself, before stopping to consider that we are not dealing with the elements alone, but with a carefully selected (or at least it should be) combination of materials. That this combination

has proved itself in practice to be equal to the conditions just stated should, until experimenters have shown otherwise, be evidence enough to justify its adoption. How many materials in actual use of construction, and which go apparently free from criticism, are there which in themselves are weak and unfit for any use, but when in combination with, or constrained by other members, do excellent service. Yet we are just as unable to give this combination a fixed limit of strength and endurance as are we in the case at hand.

To determine a value for this tensile strength of concrete within the extreme fibre, the writer had two test beams made which were designed to be weak in concrete-resisting area between the steel members in order to facilitate this method of failure if possible. The results and conclusions drawn from these tests are given further on. To be sure, the manner of failure was as anticipated, namely by tension in the concrete between the steel members. As may be noted from the tests at the time of failure, the tensile stress in the three rods of Beam No. 1 was 55,000 pounds per square inch, and in those of Beam No. 2 at the time of failure, 64,000 pounds per square inch. From No. 1 beam, provided the resisting areas of both concrete and steel were equal, the tensile stress at the time of failure would have been one-tenth of 55,000, namely 5,500 pounds per square inch, but the resisting area of the concrete was

206 ÷ 168 in excess over that of the steel, which would reduce the ultimate stress in the concrete by the reciprocal of this ratio, namely, to 4,500 pounds per square inch. From the test of No. 2 beam, this value, by like treatment, becomes 5,200. Allowing a factor of 3.5, which is allowed in all cases, as may be seen in the explanations of tables, for safety, brings the safe allowable working stress between 1,000 and 1,500 pounds per square inch. From the tables, in all cases, the safe-working stress of the concrete in tension has been kept between 1,000 and 1,500 pounds per square inch. It is not intended to assert that the value of the tensile strength of concrete can be obtained from these two experiments. They are given merely to illustrate what may be expected, and, as they bear out common practice, they are cited as fair examples of the ordinary run of experiments, which may be carried on with the object of determining the tensile strength in view.

TEST BEAM NO. 1.

Duration of set	59 days.
Kind of cement used	Portland Alpha.
Ratio of ingredients	1–2–4.
Proportion of steel	3–$\frac{3}{4}$-inch bars.
Proportion of steel	8–$\frac{1}{4}$-inch U-bars 45° to axis of beam.
Distribution of steel	$\frac{3}{4}$-inch bars $2\frac{1}{2}$ inches from bottom.
Distribution of steel	$\frac{1}{4}$-inch bars, distances varying from 4 inches at ends to 12 inches toward middle.

Manner of applying load . . .	Gradually.
Application of load	At center of span.
One-fourth load weighed on scales.	
Deflection measured by micrometer calipers.	
Length of beam	10 feet 6 inches.
Span	10 feet 0 inches.
Width of beam	5 inches.
Depth of beam	15 inches.

Results of Test.

Total load.	Increment of load.	Deflection readings.	Average deflection readings.	Deflection readings load removed.	Average deflection readings load removed.	Total set.	Deflection recovered by removal of load.	Increment of elastic deflection.	Elastic deflection per 100 lbs. load.
0.		.0405	.0405						
		.0405							
488.	488.	.0472	.0471	.0410	.0410	.0005	.0061	.0061	.00125
		.0470		.0410					
740.	252.	.0545	.0545	.0424	.0418	.0013	.0127	.0066	.00172
		.0546		.0412					
1140.	400.	.0635	.0635	.0437	.0439	.0034	.0196	.0069	.00172
		.0635		.0440					
1748.	608.	.0678	.0682	.0425	.0425	.0020	.0257	.0061	.00147
		.0685		.0425					
2300.	552.	.0784	.0784	.0450	.0447	.0042	0337	.0080	.00147
		.0784		.0440					
3448.	1148.	.1050	.1050	.0640	.0640	.0235	.0410	.0073	.00119
		.1050		.0640					
4960.	1512.	.1300	.1297	.0675	.0687	.0282	.0610	.0200	.00123
		.1295		.0700					
5760.	800.	.1530	.1537	.0778	.0778	.0373	.0759	.0149	.00132
		.1545		.0778					
6960.	1200.	.1820	.1815	.0790	.0788	.0383	.1027	.0268	.00148
		.1810		.0785					
8160.	1200.	.2145	.2145	.0856	.0857	.0452	.1286	.0259	.00158
		.2140		.0858					

Failed under	15,000 pounds.
Manner of failure	See illustration
Average elastic deflection per 100 pounds load	.00144.
Formula for elastic beam	$D = \frac{1}{48}\frac{WL^3}{EI}$.

D = elastic deflection corresponding to load W.
L = length of span in inches.
E = modulus of elasticity
$= \frac{\text{Stress per square inch}}{\text{Strain per inch}}$
I = moment of inertia of beam section about neutral axis.

$$.00144 = \frac{1 \times 100 \times 120^3}{48 \times E \times 926}\text{; whence}$$

E = modulus of elasticity, 2,640,000.

Neutral axis (when determined as already explained) above the underside of the beam	7.4 inches.
Or, below the central axis.	1.35 inches.
Maximum bending moment $= \frac{1}{4} \times 15,000 \times 120$	450,000 inch-pounds.

Concrete.

Moment of inertia —

$$I = (\tfrac{1}{12} \times 5 \times 12.5^3) + (5 \times 12.5 \times 1.35^2) = 926.$$

Distance from neutral axis to extreme fiber or layer in compression —

Y.	7.6 inches.
$\frac{I}{Y} = \frac{926}{7.6}$	122

Ultimate compressive stress at extreme fiber or layer —

$f = \frac{450,000}{122}$	3,700 lbs. sq. in.

Steel.

I = area of section × (distance of steel to neutral axis)^2 —

$3 \times .56 \times 4.9^2$	40.4.
Y	4.9.
$\frac{I}{Y} = \frac{40.4}{4.9}$	8.23.

Ultimate tensile stress at extreme layer —

$f = \frac{450,000}{8.23}$	55,000 lbs. sq. in.

Ultimate tensile stress of concrete within this layer —

$55,000 \times \frac{1}{10} \times \frac{168}{206}$	4,500 lbs. sq. in.,

where $\frac{1}{10} = \frac{\text{modulus of elasticity of concrete}}{\text{modulus of elasticity of steel}}$

and where $\frac{168}{206} = \frac{\text{area of steel in tensile layer}}{\text{area of concrete in tensile layer}}$

Test Beam No. 2.

Duration of set	62 days.
Kind of cement used	Portland Alpha.
Ratio of ingredients	1–2–4.
Proportion of steel	3-¾″ bars.
Proportion of steel	8-¼″ U-bars, 90° to axis of beam.
Distribution of steel	¾-inch bars, 3½ inches from bottom.
Distribution of steel	¼-inch bars, distances varying from 4 inches at ends to 12 inches toward middle.
Manner of applying load . .	Gradually.
Application of load	At center of span.
One-twenty-fourth load weighed on scales.	
Deflection measured by micrometer calipers.	
Length of beam	10 feet 6 inches.
Span	10 feet 0 inches.
Width of beam	5 inches.
Depth of beam	15 inches.

Results of Test.

Total load.	Increment of load.	Deflection readings.	Average deflection readings.	Deflection readings load removed.	Average deflection readings load removed.	Total set.	Deflection recovered by removal of load.	Increment of elastic deflection.	Elastic deflection per 100 lbs. load.
0.		.2714	.2714						
		.2714							
960.	960.	.2883	.2884	.2714	.2714	.0000	.0170	.0170	.00177
		.2884		.2714					
2230.	1270.	.3304	.3304	.2868	.2872	.0158	.0432	.0262	.00194
		.3305		.2880					
2810.	580.	.3435	.3437	.2740	.2733	.0019	.0704	.0272	.00251
		.3440		.2725					
3650.	840.	.3548	.3574	.2795	.2783	.0069	.0791	.0087	.00217
		.3600		.2770					
4800.	1150.	.3815	.3817	.2895	.2880	.0166	.0937	.0146	.00195
		.3820		.2865					
5040.	240.	.4010	.4010	.2950	.2045	.0231	.1065	.0128	00212
		.4010		.2940					
5930.	890.	.4190	.4195	.2960	2945	.0000	.1250	.0195	.00211
		.4200		.2930					

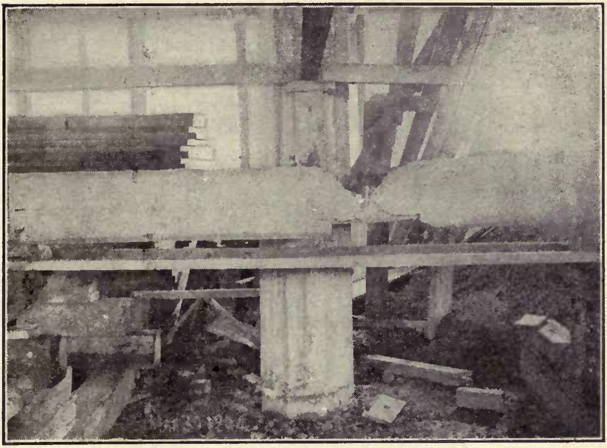
Failed under . . . . 16,200 lbs.
Manner of failure . . . . See illustration
Average elastic deflection per 100 lbs. load .00208.

$$.00208 = \frac{100 \times 120^3}{48 \times E \times 732}\text{; whence}$$

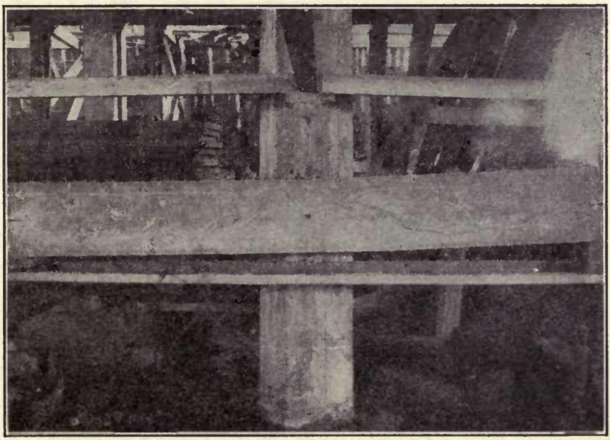
E = modulus of elasticity . . 2,420,000.

Neutral axis (above underside of beam) . . 7.95 inches.
Neutral axis (below central axis) . . . . . 1.30 inches.
Maximum bending moment

$= \frac{1}{4} \times 16,000 \times 120$ . . . . . . 480,000 in.-lbs.



Cut Showing Failure of Beam No. 1.



Cut Showing Failure of Beam No. 2.

Concrete.

Moment of inertia —

$I = (\frac{1}{12} \times 5 \times 11.5^3) + (5 \times 11.5 \times 1.30^2)$ . . . . 732.

Y . . . . 7.05 inches.

$\frac{I}{Y} = \frac{732}{7.05}$ . . . . 104.

Ultimate compressive stress at extreme layer —

$f = \frac{480,000}{104}$ . . . . 4610 lbs. sq. in.

Steel.

$I = 3 \times 56 \times 4.45^2$ . . . . 33.4.

Y . . . . 4.45.

$\frac{I}{Y}$ . . . . 7.5.

Ultimate tensile stress at extreme layer —

$f = \frac{480,000}{7.5}$ . . . . 64,000 lbs. sq. in.

Ultimate tensile stress of concrete within this layer —

$= 64,000 \times \frac{1}{10} \times \frac{168}{206}$ . . . . 5,200 lbs. sq. in.

Remarks Concerning Tests No. 1 and No. 2.

These tests were carried out, as the cuts will clearly illustrate, by applying a central load at the center of the span by means of a screw-jack, and determining same by means of allowing a portion of same to be weighed upon a set of scales. The deflection readings were obtained by fixing a pair of micrometer calipers to the beams at the center of the span, and by taking successive read-

ings when the screw of same just came into contact with a steel piano wire stretched across pins set into the ends of the beam at the neutral axis, and directly over the supports. The contact was determined more closely by allowing the micrometer screw to make an electric circuit through the piano wire, two dry cells, and an induction ringer.

CUT SHOWING ARRANGEMENT OF APPARATUS FOR CONDUCTING TESTS No. 1 AND No. 2.

All readings were checked by two individuals to .0005 or .0010 of an inch.

It will be noted that in working up the results, no attention was paid to the concrete below the tension layer of steel, and hence in Beam No. 1 the effective depth was 12.5 inches, while in Beam

No. 2 only 11.5 inches, instead of the nominal depth of 15 inches as given.

As before stated, these beams were designed to be weak in concrete-resisting area in the tensile layer between the steel, expecting this manner of failure. In one case it was clearly marked, and in the other, the first indications were tending in this direction, but which ultimately developed into a shear-crack, the failure being a compromise between the two.

Conclusions Concerning Tests.

It may be observed, by referring to the sets of deflection readings given, that like successive increments of load did not produce like increments of deflection. On the other hand, each successive like increment of load produced a deflection 10 to 30 per cent in excess over the preceding increment of deflection.

Floor Test No. 1.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
1A	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	36.5	.0310	.0850	....	....	....
	137.	.1638	.1195	....	....	....
	249.	.2412	.0969	....	....	....
	249.	.3700	.1484	....	....	....
	280.	.4607	.1648	.4688	.3438	.1250
1A–1B	Lin. ft.	Inches.	Lin. ft.			
	822.5	.0780	.0095	....	....	....
	3170.	.0938	.0296	....	....	....
	4365.	.1300	.0298	....	....	....
	4365.	.1720	.0394	....	....	....
	5600.	.2750	.0492	.2750	.2230	.0520
1B	Sq. ft.	Inches.	Sq. ft.			
	63.7	.0360	.0390	....	....	....
	249.	.1260	.0506	....	....	....
	312.	.1585	.0509	....	....	....
	312.	.2065	.0662	....	....	....
	420.	.4888	.1165	.5900	.3340	.2560
1B–1C	Lin. ft.	Inches.	Lin. ft.			
	1265.	.0620	.0049	....	....	....
	3713.	.0940	.0025	....	....	....
	4635.	.1300	.0028	....	....	....
	4635.	.1700	0037	.2800	.1300	.1500
	6300.	.2800	.0045	....	....	....
1C	Sq. ft.	Inches.	Sq. ft.			
	125	.0522	.0418	....	....	....
	245.	.1175	.0480	....	....	....
	303.	.1475	.0488	....	....	....
	303	.2125	.0702	....	....	....
	420	.4950	.1180	.5900	.3560	.2340

Floor Test No. 2.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
2C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	205.	.1190	.0580	....	....	....
	250.	.1428	.0571	....	....	....
	250.	.1962	.0785	....	....	....
	250.	.2241	.0898	.2450	.2031	.0419
1C–2C	Lin. ft.	Inches.	Lin. ft.			
	2060.	.0469	.0228	....	....	....
	2500.	.0780	.0313	....	....	....
	2500.	.0780	.0313	....	....	....
	2500.	.0780	.0313	0780	....	....
2C–3C	Lin. ft.	Inches.	Lin. ft.			
	2060	.011	.0053	....	....	....
	2500.	.033	.0132	....	....	....
	2500.	....	....	....	....	....
	2500.	.035	.0140	.0700	....	....
2C-2B	Sq. ft.	Inches	Sq. ft.			
	125.	.080	.0640	....	....	....
	125.	.080	.0640	....	....	....
	125	.094	.0753	....	....	....
	125.	.125	.1000	.1250	....	....

Floor Test No. 3.

3C	Sq. ft.	Inches.	Sq. ft.			
	....	.032	....	....	....	....
	....	.057	....	....	....	....
	250.	248	.0835	....	....	....
	250.	.255	.1020	.2550	.2550	.0000

Floor Tests No. 3. — Continued.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
3C–4C	Lin. ft.	Inches.	Lin. ft.	Inches.	Inches.	Inches.
	1500.	.046	.0370	....	....	....
	1500.	.062	.0414	....	....	....
	1500.	.078	.0520	....	....	....
	2500.	.078	.0312	....	....	....
	3750.	.094	.0251	....	....	....
	5000.	.209	.0418	.2090	.1630	.0460
3C–3B	Sq. ft.	Inches	Sq. ft.			
	1250.	.120	.0960	....	....	....
	1250.	.125	.1000	.1250	.1250	.0000

Floor Test No. 4.

4C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	1250.	.1075	....	....	....	....
	2500.	.1938	....	....	....	....
	2500.	.2250	....	....	....	....
	2500.	....	....	.2700	.2550	.0150
4C–5C	Lin. ft.					
	1250.	.0150	....	....	....	....
	1250.	.0400	....	....	....	....
	5000.	.1500	....	....	....	....
	5000.	.1700	....	....	....	....
	2500.	....	....	.1700	....	....
4C–4B	Sq. ft.					
	60.	.0320	....	....	....	....
	125.	.0900	....	....	....	....
	125.	.0480	....	....	....	....
	125.	.0650	....	.0900	....	....

Floor Test No. 5.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
5C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	250.	.2500	.1030	....	....	....
	250.	.2500	.1030	....	....	....
	250.	.2812	.1200	....	....	....
	250.	.3200	.1280	....	....	....
	250.	.2187	....	....	....	....
	250.	.1900	....	.3180	.3180	.000
5C–6C	Lin. ft.		Lin. ft.			
	2500.	.010	.0040	....	....	....
	2500.	.032	.0128	....	....	....
	3750.	.062	.0165	....	....	....
	5000.	.095	.0190	....	....	....
	5000.	.218	.0436	....	....	....
	5000.	.156	.0312	....	....	....
	2500.	.010	.0400	.2180	.1830	.035
5C–5B	Sq. ft.		Sq. ft.			
	125.	.050	.040	....	....	....
	125.	.093	.0743	....	....	....
	125.	.130	.1041	....	....	....
	125.	.124	.0991	....	....	....
	125.	.069	....	....	....	....
	125.	.050	....	.1300	.1300	.000

Floor Test No. 6.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
6C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	125.	....	....	....	....	....
	250.	.2180	....	....	....	....
	250.	.2300	....	....	....	....
	250.	.2530	....	....	....	....
	250.	.2480	....	....	....	....
	250.	.2300	....	.3250	.2550	.0700
6C–7C	Lin. ft.					
	2500.	.1250	....	....	....	....
	2500.	.1250	....	....	....	....
	5000.	.2750	....	....	....	....
	5000.	.1900	....	....	....	....
	5000.	.1600	....	....	....	....
	5000.	.2450	....	....	....	....
	2500.	.1950	....	.2450	.0890	.1560
6C–6B	Sq. ft.					
	62.	.0310	....	....	....	....
	125.	.0650	....	....	....	....
	125.	....	....	....	....	....
	125.	.0940	....	....	....	....
	125.	.1000	....	....	....	....
	125.	......	....	.1000	.1000	.0000

Floor Test No. 7.

7C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	125.	.1000	....	....	....	....
	250.	.2200	....	....	....	....
	250.	....	....	....	....	....
	250.	.2812	....	....	....	....
	250.	.2812	....	....	....	....
	250.	....	....	....	....	....
	250.	.2770	....	.2812	.1512	.1300

Floor Test No. 7. — Continued.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
7C–8C	Lin. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	1250.	.0570	....	....	....	....
	2500.	.0900	....	....	....	....
	2500.	.0980	....	....	....	....
	2500.	.1200	....	....	....	....
	3750.	.1562	....	....	....	....
	5000.	.1900	....	....	....	....
	5000.	.1500	....	....	....	....
	2500.	.1600	....	.1900	.1330	.0570
7C–7B	Sq. ft.					
	62.0	.1150	....	....	....	....
	125.	.1300	....	....	....	....
	125.	.1900	....	....	....	....
	125.	.1800	....	....	....	....
	125.	.1900	....	....	....	....
	125.	.1850	....	....	....	....
	125.	....	....	.1900	.0400	.1500

Floor Test No. 8.

8C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	125.	.0500	.0560	....	....	....
	250.	.2750	.1025	....	....	....
	250.	....	....	....	....	....
	250.	.3075	.1230	.3075	.1875	.1200
8C–9C	Lin. ft.		Lin. ft.			
	1825.	.0310	.0169	....	....	....
	5000.	.1560	.0276	....	....	....
	5000.	.2300	.0460	....	....	....
	5000.	.3700	.0740	....	....	....
	2500.	.2700	....	.3700	.2175	.1525
8C–8B	Sq. ft.		Sq. ft.			
	62.	.092	.148	....	....	....
	125.	.3125	.250	....	....	....
	125.	.3075	.246	....	....	....
	125.	....	....	.3125	.0650	.2475

Floor Test No. 9.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
9C	Sq. ft.	Inches,	Sq. ft.	Inches.	Inches.	Inches.
	62.	.093	.150	....	....	....
	250.	.234	.094	....	....	....
	250.	.247	.098	....	....	....
	250.	.345	.138	....	....	....
	250.	.329	.132	.345	.160	.185
9C–10C	Lin. ft.		Lin. ft.			
	620.	.015	.024	....	....	....
	2500.	.030	.012	....	....	....
	5000.	.129	.0258	....	....	....
	5000.	.281	....	....	....	....
	5000.	.219	.0438	.219	.074	.145
9C–9B	Sq. ft.		Sq. ft.			
	31.	.063	.201	....	....	....
	125.	.205	.164	....	....	....
	125.	.234	.187	....	....	....
	125.	.313	.250	....	....	....
	125.	.290	.232	.313	.037	.276

Floor Test No. 10.

10C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	125.	.189	.1514	....	....	....
	250.	.380	.1520	....	....	....
	250.	.370	.1480	.3800	.0675	.3125

Floor Test No. 10.—Continued.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
10C–11C	Lin. ft.	Inches.	Lin. ft.	Inches.	Inches.	Inches.
	2500.	....	....	....	....	....
	2500.	.375	.0750	....	....	....
	2500.	.406	.0810	....	....	....
	2500.	.348	.1394	.4060	.1270	.2790
10C–10B	Sq. ft.		Sq. ft.			
	62.	.156	.252	....	....	....
	125.	.284	.227	....	....	....
	125.	.268	.214	.2840	.0000	.2840

Floor Test No. 11.

11C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	125.	.1175	.0940	....	....	....
	250.	.4375	.175	....	....	....
	250.	.4688	.1876	....	....	....
	250.	.5600	.2220	.5600	.1200	.4400
11C–12C	Lin. ft.		Lin. ft.			
	1250.	.1875	.150	....	....	....
	2500.	.2188	.0877	....	....	....
	3750.	.3080	.0821	....	....	....
	5000.	.4640	.0926	.4640	.0340	.4300
11C–11B	Sq. ft.		Sq. ft.			
	62.	.2790	.455	....	....	....
	125.	.3280	.263	....	....	....
	125.	.4040	.324	....	....	....
	125.	.4970	.397	.4970	.0000	.4970

Floor Test No. 12.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
12C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	125.	.2180	.1742	....	....	....
	250.	.5050	.2020	.5050	.1925	.3125
12C–13C	Lin. ft.		Lin. ft.			
	2500.	.0660	.0264	....	....	....
	5000.	.4690	.0938	.4690	.0940	.3750
12C–12B	Sq. ft.		Sq. ft.			
	62.	.0940	.1516	....	....	....
	125.	.3780	.3022	.3870	.0795	.3075

Floor Test No. 13.

13C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	125.	.1230	.0984	....	....	....
	250.	.5800	.2320	.5800	.2360	.3440
13C–14C	Lin. ft.		Lin. ft.			
	1250.	.1510	.1210	....	....	....
	5000.	.5780	.1158	....	....	....
	2500.	.5630	....	.5780	.2950	.2830
13C–13B	Sq. ft.		Sq. ft.			
	62.	.0625	.1010	....	....	....
	125.	.4687	.3760	.4687	.0987	.3700

Results of Tests.

No. of bays.	Location in bay.	1 Average of greatest deflection.	2 Average of least deflection.	Average of 1 and 2.	3 Average greatest deflection recovered.	4 Average least deflection recovered.
1B, 1C, 2C, 3C, 4C, 5C, 6C, 7C, 8C, 9C, 10C.	Center 250 lbs. sq. ft.	....	.3117	......	....	....
1A, 11C, 12C, 13C.	Center 250 lbs. sq. ft.	.5160	....	......	....	....
Average.	....	.... ..	....	.4139	....	....
1A, 1C, 2C, 3C, 4C, 5C, 6C, and 13C,	Center 250 lbs. sq. ft.	....	....	......	.2552	....
1B, 7C, 8C, 9C, 11C, and 12C.	Center 250 lbs. sq. ft.	....	....	......	....	.1687
Average.	....	....	....	......	....	....
3C-4C, 1A-1B, 4C-5C, 5C-6C, 8C-9C, 13C-14C.	Girder 5000 lbs. lin. ft.	....	....	......	.2046	....
1B-1C, 6C-7C, 7C-8C, 9C-10C, 10C-11C, 12C-13C.	Girder 5000 lbs. lin. ft.	....	....	......	....	.1034
Average.	....	....	....	......	....	....
3C-3B, 5C-5B, 6C-6B, 13C-13B.	Side 125 lbs. sq. ft.	....	....	.....	.1134	....
7C-7B, 8C-8B, 9C-9B, 10C-10B.	....	....	....	......	....	.0554
Average.	....	....	....	......	....	....

Results of Tests. —Continued.

Average of 3 and 4.	5 Ratio $\frac{WL^3}{DEI}$ from 3 Calling $E =$ 3,000,000.	6 Ratio $\frac{WL^3}{DEI}$ from 4	Average of 5 and 6.	7 E from 5 calling $\frac{WL^3}{DEI} = \frac{5}{384}$	8 E from 6 calling $\frac{WL^3}{DEI} = \frac{5}{384}$	E Average of 7 and 8.	E Average of column 6.
......	....	....	......	....	....	....	....
......	....	....	......	....	....	....	....
......	....	....	......	....	....	....	....
......	$\frac{1}{112}$	....	......	2,060,000	....	....	....
......	....	$\frac{1}{168}$	......	....	1,375,000	....	....
.2120	....	....	$\frac{1}{140}$	....	....	1,650,000	....
......	$\frac{1}{41.2}$	....	......	5,590,000	....	....	....
......	....	$\frac{1}{81.7}$	......	....	2,820,000	....	....
.1540	....	....	$\frac{1}{61.5}$	....	....	3,750,000	....
......	$\frac{1}{127}$	....	......	1,820,000	....	....	....
......	....	$\frac{1}{260}$	......	....	888,000	....	....
.0844	....	....	$\frac{1}{194}$	....	....	1,190,000	2,197,000

Remarks Concerning Floor Tests.

These tests were carried out upon a floor composed of 20-foot square bays, a $3\frac{1}{4}$-inch floor, 5×12-inch beams, 18 feet 10 inches long between girders, and spaced 3-feet 5-inch centers, and 14×24-inch girders, clear span 18 feet 2 inches, all of concrete reinforced by steel rods. This floor had been erected eight months previous to the time of testing, having withstood in the meantime the effects of a severe winter, although protected as well as could be. The load consisting of sand bags, was uniformly distributed, and covered three consecutive bays at once in order to obtain the full loading over the center bay, and the girders on either side supporting same.

The loads given under column 2 are in addition to the weight of the floor itself.

Deflection readings were taken both by an engineers' level and by a multiplying lever, which recorded four times the actual deflection. The items entered under column 3 are the means of the deflection taken by both methods. Column 5 gives the greatest deflection resulting from the greatest loading recorded under column 2. Column 6 shows the amount of deflection recovered upon the removal of all the live loading. This represents the elastic deflection, and is the measure of the elasticity of the floor from which the modulus of elasticity, given under "Results of Tests," was computed.

Floor Test No. 8b. — Time of Set, 8 Months.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
8B	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	200.	.2188	.1094	....	....	....
	385.	.3750	.0975	....	....	....
	400.	.4055	.1014	.4055	.3375	.0680
8B–8C	Sq. ft.		Sq. ft.			
	100.	.1250	.1250	....	....	....
	193.	.2530	.1311	....	....	....
	200.	.2570	.1285	.2570	.1640	.0930
7B–8B	Lin. ft.		Lin. ft.			
	2000.	.0314	.0157	....	....	....
	3650.	.0625	.0171	....	....	....
	4000.	.1200	.0300	.1200	.0650	.0550
8A–8B	Sq. ft.		Sq. ft.			
	100.	.0910	.0910	....	....	....
	193.	.1850	.0960	....	....	....
	200.	.2100	.1050	.2100	.1760	.0340
8B–9B	Lin. ft.		Lin. ft.			
	2000.	.0310	.0155	....	....	....
	3650.	.0660	.0181	....	....	....
	4000.	.0890	.0223	.0890	.0740	.0150

Floor Test No. 14.

1	2.	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set.
14C	Sq. ft.	Inches.	Sq. ft.	Inches.	Inches.	Inches.
	60.	.1250	.2090	....	....	....
	150.	.2500	.1670	....	....	....
	175.	.4063	.2320	....	....	....
	200.	.5938	.2969	....	....	....
	210.	.6563	.3130	....	....	....
	220.	.7188	.3270	....	....	....
	235.	.8750	.3730	....	....	....
	250.	1.0000	.4000	....	....	....
	250.	1.0780	.4320	1.0780	.5467	.5313
13C–14C	Lin. ft.		Lin. ft.			
	2500.	.1250	....	.1250	.0940	.0310
14C–14B	Lin. ft.					
	2500.	.1300	....	.1300	.1010	.0290
14C–15C	Lin. ft.					
	2500.	.3700	....	.3700	.2100	.1600
14C–14D	Lin. ft.					
	2500.	.2813	....	.2813	.1563	.1250

Time of set, 17 days.

14C represents 8-inch flat slab, 20-foot 0-inch span each way.

13C–14C represents 14 × 24-inch girder, 20-foot 0-inch span.

14C–14B represents 14 × 24-inch girder, 20-foot 0-inch span.

14C–15C represents 22 × 8-inch girder, 20-foot 0-inch span.

14C–14D represents 22 × 8-inch girder, 20-foot 0-inch. span.

NOTE. — Cracks developed diagonally across flat slab under 175 pounds square foot. Opened badly at end of test.

Roof Test No. 22. — Time of Set, 14 Days.

1	2	3	4	5	6	7
Location in section.	Total load.	Mean deflection.	Mean increment deflection per 100 lbs. per sq. ft. or per 1000 lbs. lin. ft.	Greatest deflection.	Amount deflection recovered.	Permanent set
21A–22A	Lin. ft.	Inches.		Inches.	Inches.	Inches.
	1250.	.0000	....	....	....	....
	1100.	.0000	....	.0000	.0000	.0000
22A	Sq. ft.		Sq. ft.			
	62.5	.1860	.2980	....	....	....
	55.0	.1235	.2250	.1860	.1225	.0625
22A–23A	Lin. ft.		Lin. ft.			
	1250.	.0000	....	....	....	....
	1100.	.0000	....	.0000	.0000	.0000
21C–22C	Lin. ft.					
	1250.	.0000	....	.0000	.0000	.0000
22C	Sq. ft.		Sq. ft.			
	62.5	.0700	.1120	.0700	.0700	.0000
22C–23C	Lin. ft.					
	1250.	.0000	....	.0000	.0000	.0000

The roof was composed of a 20-foot square bay, $2\frac{1}{2}$ inches thick, carried by 2.5 × 12-inch beams, 19-foot 4-inch span, spaced 3-foot 2-inch centers, which, in turn, were carried by 8 × 24-inch girders, 19-foot 0-inch clear span.

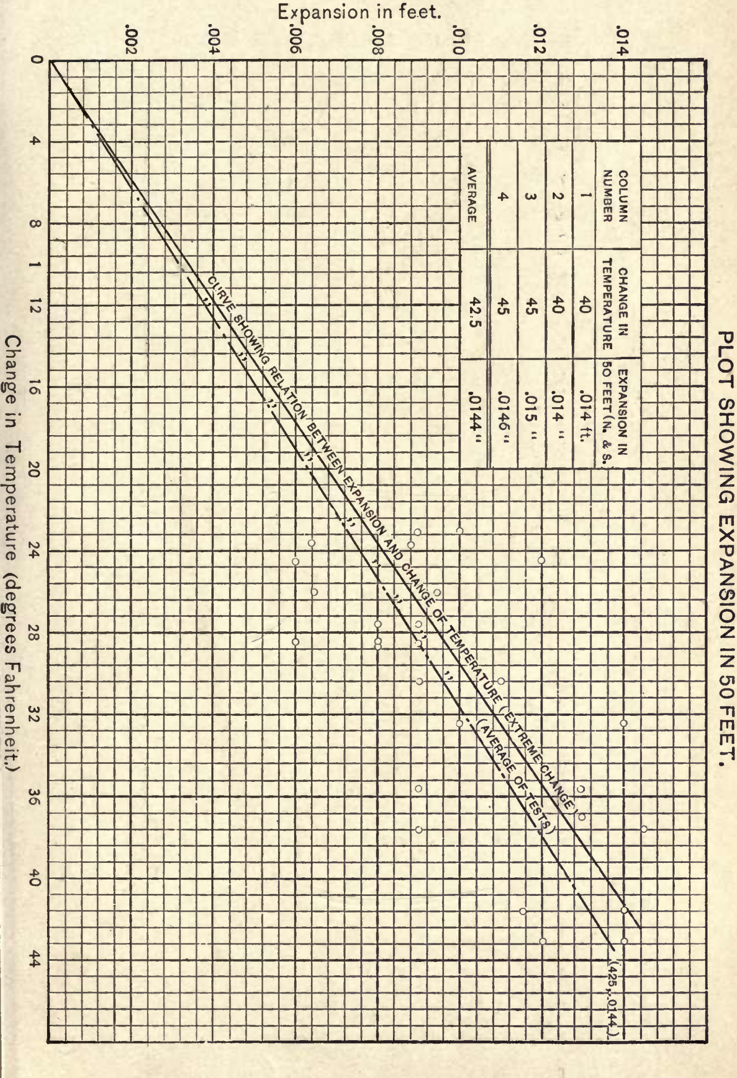
Remarks Upon Plots.

The following three plots are inserted to show the results of various tests taken to determine the effect of temperature upon reinforced concrete as regards expansion and contraction. The first gives the change in length of a section of floor fifty feet long for corresponding changes of temperature. The second plot gives the change in thirty feet, while the third is a combination of the two reduced to a section ten feet long.

At first glance the results may seem to vary greatly from the mean or from the curves drawn to represent the relationship between expansion and temperature. This is due to plotting the ordinates to a large scale, which shows up the irregularities to a great extent in the first two plots. However, when these are reduced to Plot 3, the curves drawn are seen to fairly represent the average relationship desired.

The curves shown by full lines were plotted between points *o*, *o*, and the average abscissæ, and ordinates taken from the tables shown on Plots 1 and 2, which give the extreme changes. On the other hand, the curves shown by broken lines give the average results of the tests.

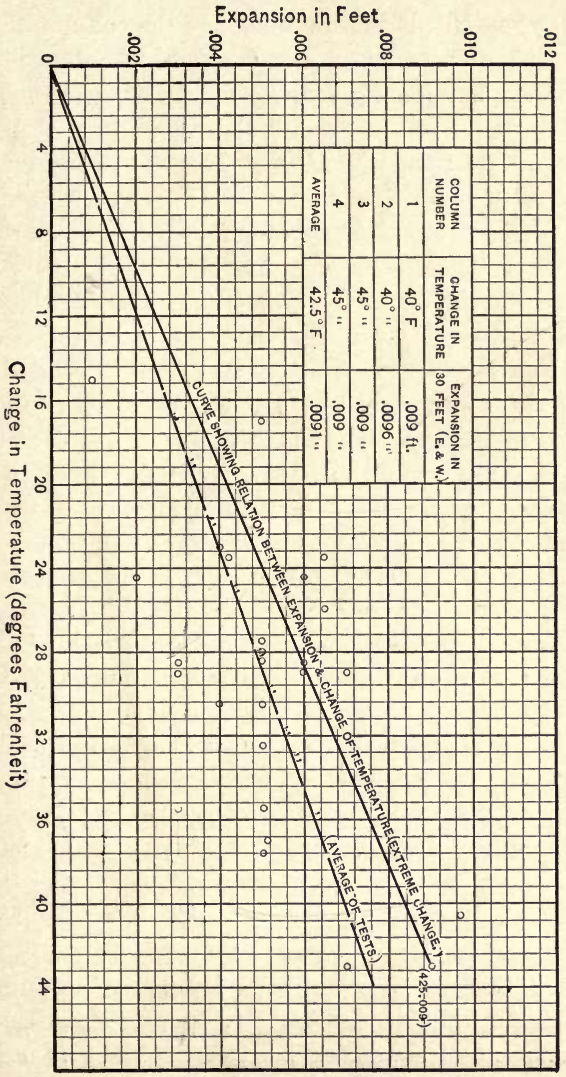
The method of carrying on these tests was as follows: A set-up point was located, so that it could be produced at any time. From this point a transit line was produced upon the corner column of a building between the ground and the


PLOT SHOWING EXPANSION IN 50 FEET.
Expansion in feet.
.002
.004
.006
.008
.010
.012
.014
0
4
8
12
16
20
24
28
32
36
40
44
Change in Temperature (degrees Fahrenheit.)
COLUMN NUMBER
CHANGE IN TEMPERATURE
EXPANSION IN 50 FEET (N. & S.)
1
40
.014 ft.
2
40
.014 "
3
45
.015 "
4
45
.0146 "
AVERAGE
42.5
.0144 "
CURVE SHOWING RELATION BETWEEN EXPANSION AND CHANGE OF TEMPERATURE (EXTREME CHANGE)
(AVERAGE OF TESTS)
(42.5, .0144)

underside of the first floor, and the same scratched in with a fine line by a knife. At the same time the temperature was taken of the atmosphere in the vicinity of the first floor, and also of the ground near the base of the column. These temperatures were assumed to be the temperatures of the first floor and of the column footing respectively. Since these tests were not taken immediately after a sudden change of temperature, the concrete of the first floor had time in all cases to assume the outside temperature as nearly as might be, and hence, the assumption just made cannot be far from the true values.

To make a test, the same transit was accurately set up over the point already determined, a foresight was taken at either the top or bottom extremity of the knife scratch, and the line produced at the other extremity of the scratch, and any difference between the new and original lines noted. At the same time temperatures were taken both of the outside atmosphere and of the ground. From these the change of temperature between the first floor and the column footing could be determined, and this change of temperature resulted in a change of length of the first floor, as just determined by the difference between the new and original lines.

These tests were carried on at a time of the year when the variation of the temperature of the ground was very slight, and could have been neglected without affecting the value of the tests



COLUMN NUMBER	CHANGE IN TEMPERATURE	EXPANSION IN 30 FEET (E. & W.)
1	40° F	.009 ft.
2	40° "	.0096 "
3	45° "	.009 "
4	45° "	.009 "
AVERAGE	42.5° F	.0091 "

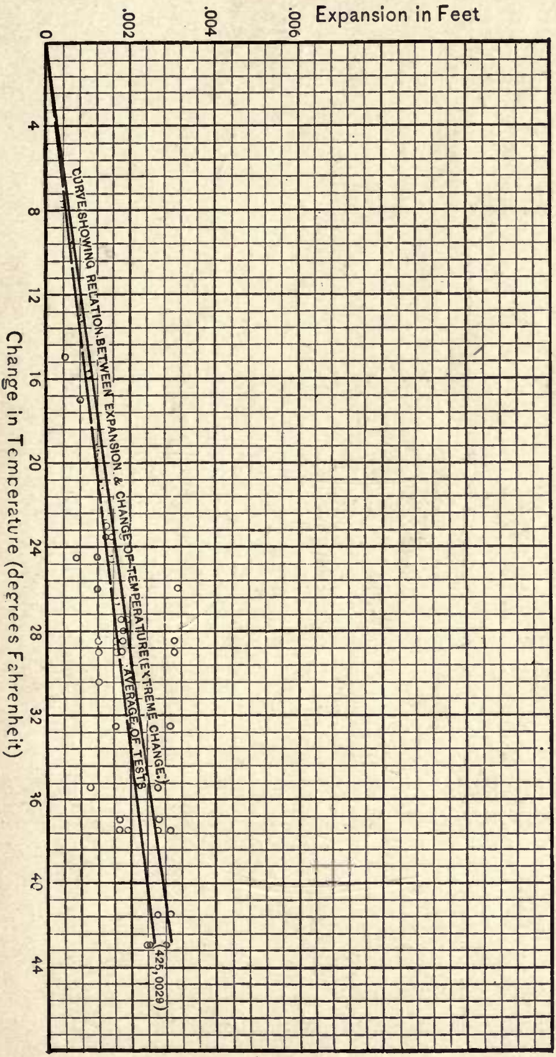
PLOT SHOWING EXPANSION IN 30 FT.

to any extent, because the base of the column, the footing of same, and the curtain walls between this column footing and its neighbors, were surrounded by frozen earth and ice. However, any slight variation, when it was known not to be in error, was allowed for.

Readings were taken on the four corner columns, and in both directions, namely, east-west and north-south, on each of the four columns. This required eight set-ups. The building in question had an expansion joint fifty feet from the ends in a north-south direction, and was sixty feet wide, thus allowing a length for expansion in an east-west direction of thirty feet for each of the two corner columns at either end.

Conclusions.

By referring to the combined plot, it may be seen that the data contained there is sufficient to determine the coefficient of expansion. Take the point, for instance, whose coördinates are 42.5, .0029. This means that for a change in temperature of 42.5°, there was a corresponding change in length of .0029 feet per ten feet, or .00029 feet per one foot. The corresponding change in length per one degree may be represented by the expression .00029 ÷ 42.5 equals .00000682, which is the coefficient of expansion. By using the point whose coördinates are 42.5, .0026, the resulting coefficient of expansion becomes .00000601. The



COMBINED PLOT SHOWING EXPANSION IN 10 FEET

mean of the two is .00000642, and this value is used in the following.

In temperate climates a change of temperature of 70° F. may be considered a maximum either way from the temperature under which the original setting ordinarily takes place. This change would cause a strain of $.00000642 \times 70 = .000448$ inches in the concrete, and, since the coefficient of expansion of steel is .00000657, the strain in the steel would be .000459 inches, or practically the same as that in the concrete. This strain causes a stress of 13,780 pounds per square inch in the steel, calling the modulus of elasticity 30,000,000.

To determine the percentage of metal required for a change of 70° when there is developed in the steel a stress not greater than the elastic limit, which may be considered 52,000 pounds per square inch with a high carbon steel, we may proceed as follows: The amount of stress which may be brought to bear upon the steel by the concrete, when the same has reached its ultimate tensile stress, $= 52{,}000 - 13{,}780 = 38{,}220$ pounds per square inch. The ultimate tensile stress of the concrete may be considered to be 300 pounds per square inch. Hence the remaining stress of 38,220 pounds per square inch would resist the stress of $38{,}220 \div 300 = 127$ square inches of concrete stressed to 300 pounds per square inch.

In order to develop the ultimate stress in a square foot section of concrete would require a section of steel that would resist the stress of

72 square inch concrete after the same has reached its ultimate tensile stress. Under this condition the steel section would offer half the resistance to elongation, while the concrete would offer the other half, and the square foot section, which might be treated as 72 square inches of concrete and its equal of steel, each stressed to its elastic limit, or as 144 square inches of concrete stressed to 300 pounds per square inch, its ultimate tensile stress. Consequently, since 1 square inch of steel stressed to 52,000 pounds per square inch is equivalent to 127 square inches of concrete stressed to 300 pounds per square inch in tension, to care for one square foot of concrete, considered as 72 square inches of concrete and its equivalent of steel, would require $72 \div 127 = .57$ or practically .6 square inches of steel.

PART III.

DESIGNS OF CONCRETE STRUCTURES.

TABLE I.

Description of Table I.

In using Table I, all that is required to be known, in order to design the beam or girder, is the maximum bending moment. When this is known, either in inch-pounds or foot-pounds, pick out the next larger value in column 5, if in inch-pounds, or in column 6, if in foot-pounds, provided the designer will accept a factor of safety of 3.5 as ample; if not, use columns 7 or 8 in a like manner, which allow for a factor of safety of 5. From the location of the proper moment to fit the case at hand, by following horizontally to the left, column 1 will give the size of beam as far as concrete is concerned. This size includes the concrete protection below the steel tension members and the base of the floor, but does not allow for the top 1-inch finish. In other words, the depth of the beam given in column 1, which in all cases is the second of the two dimensions, takes into account the entire depth save 1 inch, which is to be added to the top of floor for wear.

Column 5 gives the total moment that the size can withstand allowing the specified factor of safety, which total includes the moment due to

the dead load of the concrete itself as well as that due to the live load.

To facilitate computations, in column 2 is given the weight per lineal foot of the beam, from which the moment, due to the dead load, can be ascertained by taking a trial size.

Column 4 gives the ratio of the moment of inertia of the section about the neutral axis which is lettered I, to the distance of the upper layer of fibers, which layer in all cases has the most stress to withstand by compression, from the neutral axis, lettered y. This ratio, of course, is the same as that of the safe maximum bending moment, expressed in inch-pounds, to the safe allowable stress per square inch of the concrete in compression. It is given more as a check than from any practical use it bears to the use of the table.

Under column 5 is given the section of the steel member or members, which, by virtue of being embedded in the concrete, is able to withstand the tensile stress in the worst layer resisting tension, allowing the same factor of safety as was allowed in the concrete, namely, 3.5 or 5.0. Of course, as long as the extreme layer remains strained below the elastic limit, all layers approaching the neutral axis, which must necessarily be less strained, are obliged to remain intact, and there can come undue stress on these layers only after the extreme layer has been strained beyond the elastic limit. Accordingly, in designing the

beam for tension, it is necessary only to put enough steel in the extreme layer to withstand the tension in that layer, allowing the proper factor of safety. All layers approaching the neutral axis will have their corresponding tensile stresses properly resisted by the concrete, as long as the extreme layer remains intact. The steel section as here given in all cases, is designed to be placed in the beam or girder with its lowermost part, at the center of the span, just 1 inch above the underside of the beam, but never less, in order to be sufficiently protected by the concrete in case of fire. At the center of the span, the location should never be more than 1 inch above the bottom, without making allowance for the lessening of the moment of resistance.

Under column 10 is given what is termed by the heading, "The Proper Size of Bars." This, at first thought, may appear uncalled for, as long as the area of section is given, but after reading what is outlined under the "Tensile Strength of Concrete," in another section, the reason may appear. Briefly, in cases where it is required to use two or more rods to equal the section, a number might be selected leaving very little concrete between the different steel members in the layer along with them. This total resisting area of the concrete between the rods might, in extreme cases, be so reduced that it would be incapable of transferring the tensile stress from member to member, because of an excessive stress per square

inch upon the area. In such a case it might be practically impossible to work in the concrete between the steel members without leaving voids, or allowing the members to rub together with only a film, if any, of concrete between. With this in view, the items under this column were so selected as to leave sufficient area of concrete between the steel so that this latter would not have to carry over 1,000 to 1,500 pounds per square inch in tension, and at the same time allow space to properly work in the concrete between the rods. This area is given under column 12, and the corresponding tensile stress per square inch under column 13.

Column 11 gives the distance below the center of gravity of the section to the neutral axis. This is determined after the steel section is known, and the rods selected, by substituting an area of concrete, which, when placed at the location of the steel, would give the same tensile resisting power at the extreme layer as does the section of steel. This area is considered attached to the beam so that its depth is equal to one side of the square rod or rods, and its width ten times the total width of the rod or rods — ten times because the modulus of elasticity of the steel is ten times that of the concrete. By so doing, we obtain an inverted T section, and it remains, in order to determine the neutral axis, only to determine the center of gravity of this section by the method of moments.

It has just been stated that the steel section was so designed as to be protected by 1 inch of concrete at the center of the span. This was fixed at 1 inch by balancing up two important factors, each directly opposed to the other. For instance, to render the beam or girder fire resisting, it is well to have the steel members thoroughly protected from below by concrete, which tends to have the tension members approach the neutral axis. On the other hand, in order to obtain the greatest moment of resistance in tension, the tendency is to have the tension members approach the underside of the beam or girder. Along this same reasoning, in order to prevent hair cracks, caused by excessive tension due to deflection, across the underside of the beam which, although they do not to any extent effect the strength of the beam, are very unsightly, the tendency is to have the tension members approach the underside of the beam, since this adds to the stiffness of the beam, and thereby lessens the tension in the concrete below the steel. By equating these factors, judgment will fix the location, especially at the center of the span, about 1 inch from the underside of the tension members to the underside of the beam or girder.

Finally, to give an outline how the values $\frac{I}{Y}$ and the safe allowable resisting moments, both of compression and of tension, were deduced, the following routine is given:

Let M = Maximum bending moment.
b = Width of beam or a width of floor corresponding to bending moment above.
d = Depth of beam or thickness of floor down to center of tension members.

Then $\frac{M}{500} = \frac{1}{4}\frac{bd^3}{d}$ whence assuming b, d is computed.

This is a preliminary step, but after the neutral axis is located, and using the value of d just found, the fiber stress at top fiber figures about 850 pounds per square inch. Calling the ultimate compressive stress 3,000 pounds per square inch, which should be attained in a 1–2–4 or a 1–1½–3 mixture, the stress just found gives a factor of safety of 3½.

The next step is to find the area of steel which, when taking all the stress in the worst position of tension, takes a stress of 15,000 pounds per square inch. This is figuring a factor of safety of 3.5, with an ultimate stress of 52,000 or 53,000. To do this, I assume the neutral axis to be from 1.5 to 2.0 inches below the center of gravity, and figure the area of the steel thus:

$$\frac{M}{15{,}000} = \frac{ah^2}{h}.$$

Where a = area of steel.
h = distance from center line of steel to neutral axis assumed above.

With this area of steel, the neutral axis can be located by taking moments, after transposing the area of steel into an area of concrete, etc., as stated before. If this location does not come sufficiently near 1.5 or 2.0 inches below the center of gravity of the section to fulfil the assumption previously made, use this value to determine h in in the last formula

(Namely $\frac{M}{15,000} = ah$), and solve for a again.

This new value of a will probably not change the location of neutral axis, found previously, enough to effect the results. Now we are able to figure the fiber stress of the concrete in compression and so check the 850 pounds per square inch with the sizes we had determined, or else fix new sizes to give no more than 850 pounds per square inch for the concrete in compression.

TABLE I. — *Beams and Girders, Single Spans Supported at Ends.*

1	2	3	4	5	6	7	8	9	10	11	12	13
Size of beam or girder.	Weight per lineal foot.	Area of section.	Values of ratio. $\frac{I}{y}$.	Safe bending moment. Factor of safety = 3.5.		Safe bending moment. Factor of safety = 5.0.		Area of steel in tension side.	Number and size of rods.	Location of neutral axis below central axis.	Area of concrete in tensile layer.	Tensile stress in concrete in tensile layer.
Inches.	Lbs.	Sq. in.		In. lbs.	Ft. lbs.	In. lbs.	Ft. lbs.	Sq. in.		Inches.	Sq. in.	Lbs. sq. in.
$2\frac{1}{2}\times 6$	15.	15	9.46	7,800	650	5,460	450	.47	1– $\frac{11}{16}$″	.9	1.82	390
$2\frac{1}{2}\times 8$	20.	20	18.60	15,350	1,280	10,745	896	.52	1– $\frac{3}{4}$ ″	1.03	1.31	590
$2\frac{1}{2}\times 10$	25.	25	30.60	25,250	2,100	17,675	1,470	.65	1– $\frac{13}{16}$″	1.11	1.36	715
$2\frac{1}{2}\times 12$	30.	30	45.8	37,800	3,150	26,460	2,210	.66	1– $\frac{13}{16}$″	1.17	1.36	740
3×6	18.8	18	11.4	9,380	780	6,566	550	.48	1– $\frac{3}{4}$ ″	.70	1.69	425
3×8	25.	24	21.7	18,450	1,540	12,915	1,080	.59	1– $\frac{13}{16}$″	.90	1.78	500
3×10	31.2	30	36.4	30,000	2,500	21,000	1,750	.66	1– $\frac{13}{16}$″	.98	1.78	555
3×12	37.5	36	55.2	45,500	3,790	31,850	2,650	.78	1– $\frac{7}{8}$ ″	1.10	1.86	640
3×14	43.8	42	77.0	63,500	5,290	44,450	3,700	.88	1– $\frac{15}{16}$″	1.18	1.93	685
4×8	33.3	32	29.7	24,500	2,040	17,150	1,430	.75	1– $\frac{7}{8}$ ″	.80	2.74	410
4×10	41.7	40	49.1	40,500	3,380	28,350	2,360	.86	1– $\frac{15}{16}$″	.85	2.87	450
4×12	50.0	48	73.4	60,500	5,040	42,350	3,530	.99	1–1 ″	.93	3.00	495

4×14	58.2	56	102.5	84,500	7,040	59,150	4,930	1.14	1–1 1/16″	1.03	3.13	545
4×16	66.6	64	136.4	112,500	9,380	78,750	6,560	1.26	1–1 1/8 ″	1.03	3.24	585
5×10	52	50	61.4	50,650	4,220	35,455	2,950	1.15	2– 3/4 ″	1.06	2.62	660
5×12	62.6	60	91.7	75,600	6,300	52,920	4,410	1.32	2– 13/16″	1.19	2.74	725
5×14	73.2	70	128.0	105,500	8,790	73,850	6,150	1.46	2– 7/8 ″	1.20	2.84	770
5×16	83.5	80	170.5	140,500	11,710	98,350	8,200	1.68	2– 15/16″	1.42	3.00	840
5×18	93.6	90	218.5	180,000	15,000	126,000	10,500	1.86	2–1 ″	1.55	3.00	930
5×20	104.6	100	273.0	225,500	18,790	157,850	13,150	2.06	2–1 1/16″	1.73	3.06	1010
6×12	75	72	110.5	91,000	7,580	63,700	5,310	1.53	2– 7/8 ″	1.03	3.72	711
6×14	87.7	84	154	127,000	10,580	88,900	7,410	1.78	2– 15/16″	1.25	3.87	690
6×16	99.7	96	204	168,500	14,040	117,950	9,820	1.97	2–1 ″	1.28	4.00	740
6×18	112.5	108	213	217,000	18,080	151,900	12,660	2.19	2–1 1/16″	1.40	4.11	800
6×20	125.2	120	328	270,500	22,540	189,350	15,780	2.40	2–1 1/8 ″	1.48	4.22	850
6×22	138	132	403	332,500	27,710	232,750	19,400	2.62	2–1 1/8 ″	1.51	4.22	930
6×24	150	144	483	398,500	33,210	278,950	23,250	2.84	2–1 1/4 ″	1.63	4.62	925
7×14	102.5	98	179.5	148,000	12,330	103,600	8,630	2.14	3– 7/8 ″	1.38	3.83	830
7×16	117.2	112	239	197,000	16,420	137,900	11,500	2.41	3– 15/16″	1.55	3.92	920
7×18	131.4	126	306	252,500	21,040	176,750	14,730	2.62	3– 15/16″	1.58	3.92	1005
7×20	146.4	140	382	315,000	26,250	220,500	18,380	2.87	3–1 ″	1.67	4.00	1080
7×22	160.4	154	470	387,500	32,280	271,250	22,600	3.11	3–1 1/16″	1.70	4.00	1170
7×24	175.0	168	563	464,500	38,710	325,150	27,100	3.27	3–1 1/16″	1.52	3.99	1230
7×26	190	182	663	547,500	45,630	383 250	31,940	3.53	3–1 1/16″	1.64	3.99	1330
7×28	204	196	775	639,500	53,290	447,650	37,300	3.77	3–1 1/8 ″	1.70	4.08	1385

TABLE I. — *Beams and Girders.* — Continued.

1	2	3	4	5	6	7	8	9	10	11	12	13
Size of beam or girder.	Weight per lineal foot.	Area of section.	Values of ratio $\frac{I}{y}$.	Safe bending moment. Factor of safety = 3.5.		Safe bending moment. Factor of safety = 5.0.		Area of steel in tension side.	Number and size of rods.	Location of neutral axis below central axis.	Area of concrete in tensile layer.	Tensile stress in concrete in tensile layer.
Inches.	Lbs.	Sq. in.		In. lbs.	Ft. lbs.	In. lbs.	Ft. lbs.	Sq. in.		Inches.	Sq. in.	Lbs. sq. in.
8×16	133	128	273	225,000	18,750	157,500	13,120	2.63	3– $\frac{15}{16}$″	1.28	4.70	840
8×18	150	144	350	288,500	24,040	201,950	16,830	2.90	3–1 ″	1.38	5.00	870
8×20	167	160	437	360,500	30,040	252,350	21,030	3.24	3–1 $\frac{1}{16}$″	1.58	5.12	950
8×22	183	176	537	442,500	36,880	309,750	25,820	3.55	3–1$\frac{1}{8}$ ″	1.68	5.20	1025
8×24	200	192	645	531,000	44,250	371,700	30,980	3.82	3–1$\frac{1}{8}$ ″	1.70	5.20	1105
8×26	217	208	757	625,000	52,080	437,500	36,460	4.09	3–1 $\frac{3}{16}$″	1.80	5.27	1160
8×28	233	224	880	725,000	60,420	507,500	42,300	4.32	3–1 $\frac{3}{16}$″	1.83	5.27	1230
8×30	250	240	1020	840,000	70,000	588,000	49,000	4.65	3–1$\frac{1}{4}$ ″	1.96	5.32	1315
8×32	267	256	1165	962,000	80,170	673,400	56,120	4.97	3–1 $\frac{5}{16}$″	2.05	5.25	1420
9×18	168.8	162	393	324,000	27,000	226,800	18,900	3.38	4– $\frac{15}{16}$″	1.6	4.92	1030
9×20	187	180	493	406,000	33,830	284,200	23,680	3.73	4–1 ″	1.73	5.00	1120
9×22	206	198	605	498,500	41,540	348,950	29,080	4.03	4–1 ″	1.75	5.00	1210
9×24	225	216	723	597,000	49,750	417,900	34,830	4.37	4–1 $\frac{1}{16}$″	1.87	5.03	1310

9×26	244	234	853	702,500	58,540	491,750	40,980	4.65	4-1$\frac{1}{16}$″	1.90	5.03	1391
9×28	262	252	970	800,000	66,660	560,000	46,660	4.73	3-1$\frac{1}{4}$ ″	1.70	6.57	1080
9×30	281	270	1148	946,500	78,870	662,550	55,210	5.22	3-1$\frac{3}{8}$ ″	1.90	6.71	1170
9×32	300	288	1248	1,082,500	90,210	757,750	63,150	5.54	3-1$\frac{3}{8}$ ″	1.95	6.71	1240
9×34	318.4	306	1487	1,227,500	102,290	859,250	71,600	5.87	3-1$\frac{7}{16}$″	2.04	6.20	1420
9×36	337.5	324	1675	1,380,000	115,000	966,000	80,500	6.16	3-1$\frac{7}{16}$″	2.07	6.20	1490
10×20	209	200	548	451,000	37,580	315,700	26,310	4.10	4-1 ″	1.67	6.00	1025
10×22	228 6	220	672	554,000	46,170	387,800	32,320	4.43	4-1$\frac{1}{16}$″	1.69	6.11	1090
10×24	250	240	805	664,000	55,330	464,800	38,730	4.90	4-1$\frac{1}{8}$ ″	1.80	6.21	1185
10×26	271	260	945	780,000	65,000	546,000	45,500	5.12	4-1$\frac{1}{8}$ ″	1.83	6.21	1235
10×28	291	280	1108	914,000	76,170	639,800	53,320	5.40	3-1$\frac{3}{8}$ ″	1.70	8.09	1000
10×30	312.5	300	1247	1,050,000	87,500	735,000	61,250	5.73	3-1$\frac{7}{16}$″	1.78	8.18	1050
10×32	334	320	1457	1,200,000	100,000	840,000	70,000	6.09	3-1$\frac{1}{2}$ ″	1.85	8.25	1110
10×34	354	340	1655	1,365,000	113,750	955,500	79,630	6.71	3-1$\frac{1}{2}$ ″	1.88	8.25	1220
10×36	375	360	1858	1,532,000	127,670	1,072,400	89,370	6.80	3-1$\frac{9}{16}$″	2.00	8.30	1230
10×38	396	380	2065	1,702,000	141,830	1,191,400	99,280	7.14	3-1$\frac{9}{16}$″	2.10	8.30	1290
10×40	416	400	2310	1,905,000	158,750	1,333,500	111,130	7.35	3-1$\frac{5}{8}$ ″	2.14	8.33	1355
11×22	252	242	740	610,000	50,830	427,000	35,580	4.97	5-1 ″	1.8	6.00	1240
11×24	275	264	833	729,000	60,750	510,300	42,530	5.36	5-1$\frac{1}{16}$″	1.92	6.12	1315
11×26	298	286	1040	858,000	71,500	600,600	50,050	5.68	5-1$\frac{1}{16}$″	1.92	6.12	1390
11×28	321	308	1216	1,002,500	83,540	701,750	58,480	6.02	4-1$\frac{1}{4}$ ″	1.90	7.50	1200
11×30	343.5	330	1415	1,157,000	96,420	809,900	67,490	6.48	4-1$\frac{1}{4}$ ″	2.10	7.50	1290
11×32	367	352	1608	1,325,000	110,420	927,500	77,290	6.65	3-1$\frac{1}{2}$ ″	1.72	9.75	1030
11×34	389	374	1820	1,500,000	125,000	1,050,000	87,500	7.07	3-1$\frac{9}{16}$″	1.85	9.86	1075

TABLE I. — *Beams and Girders.* — Continued.

1	2	3	4	5	6	7	8	9	10	11	12	13
Size of beam or girder.	Weight per lineal foot.	Area of section.	Values of ratio $\frac{I}{y}$.	Safe bending moment. Factor of safety = 3.5.		Safe bending moment. Factor of safety = 5.0.		Area of steel in tension side.	Number and size of rods.	Location of neutral axis below central axis.	Area of concrete in tensile layer.	Tensile stress in concrete in tensile layer.
Inches.	Lbs.	Sq. in.		In. lbs.	Ft. lbs.	In. lbs.	Ft. lbs.	Sq. in.		Inches.	Sq. in.	Lbs. sq. in.
11×36	412.5	396	2045	1,685,000	140,420	1,179,500	98,290	7.45	3–1$\frac{5}{8}$ "	1.90	9.95	1125
11×38	436	418	2270	1,871,000	155,920	1,309,700	109,140	7.80	3–1$\frac{11}{16}$"	2.00	9.64	1215
11×40	458	440	2537	2,092,000	174,330	1,464,400	122,030	8.24	3–1$\frac{11}{16}$"	2.04	9.64	1285
11×42	481.8	462	2805	2,312,500	192,710	1,618,750	134,900	8.61	3–1$\frac{3}{4}$ "	2.06	10.06	1285
11×44	505	484	3080	2,540,000	211,670	1,778,000	148,170	8.95	3–1$\frac{3}{4}$ "	2.06	10.06	1335
12×24	300	288	965	796,000	66,330	557,200	46,430	5.58	5–1$\frac{1}{16}$"	1.46	7.18	1165
12×26	325.5	312	1138	937,000	78,080	655,900	54,660	6.08	4–1$\frac{1}{4}$ "	1.72	8.75	1045
12×28	350	336	1328	1,095,000	91,250	766,500	63,880	6.51	4–1$\frac{5}{16}$"	1.87	8.87	1100
12×30	375	360	1530	1,262,500	105,210	883,750	73,650	6.97	4–1$\frac{5}{16}$"	1.90	8.87	1180
12×32	400	384	1745	1,440,000	120,000	1,008,000	84,000	7.33	4–1$\frac{3}{8}$ "	1.90	8.94	1230
12×34	425	408	1988	1,640,000	136,670	1,148,000	95,670	7.61	4–1$\frac{7}{16}$"	1.65	8.96	1275
12×36	450	432	2235	1,841,000	153,420	1,288,700	107,390	8.12	3–1$\frac{11}{16}$"	1.82	11.32	1080
12×38	475	456	2462	2,032,000	169,330	1,422,400	118,530	8.42	3–1$\frac{3}{4}$ "	1.90	11.81	1070
12×40	500	480	2730	2,249,000	187,410	1,574,300	131,190	8.92	3–1$\frac{3}{4}$ "	1.90	11.81	1130

12×42	525	504	2995	2,466,000	205,490	1,726,200	143,850	9.35	3-1$\frac{13}{16}$"	1.98	11.90	1180
12×44	550	528	3260	2,683,000	223,570	1,878,100	156,510	9.85	3-1$\frac{13}{16}$"	2.03	11.90	1240
12×46	575	552	3520	2,900,000	241,670	2,030,000	169,170	10.25	3-1$\frac{7}{8}$ "	2.10	11.90	1290
12×48	600	576	3850	3,175,000	264,580	2,222,500	185,210	10.75	3-1$\frac{7}{8}$ "	2.10	11.95	1350
13×26	352	338	1230	1,015,000	84,580	710,500	59,210	6.60	5-1$\frac{3}{16}$"	1.75	8.30	1190
13×28	379	364	1440	1,187,500	98,960	831,250	69,270	7.05	5-1$\frac{3}{16}$"	1.86	8.30	1275
13×30	407	390	1583	1,367,500	115,630	956,550	80,940	7.68	5-1$\frac{1}{4}$ "	1.95	8.43	1365
13×32	434	416	1890	1,560,000	130,000	1,092,000	91,000	8.08	5-1$\frac{5}{16}$"	2.12	8.43	1440
13×34	462	442	2148	1,770,000	147,500	1,239,000	103,250	8.49	5-1$\frac{5}{16}$"	2.10	8.43	1510
13×36	488	468	2420	1,995,000	166,250	1,396,500	116,380	8.92	4-1$\frac{1}{2}$ "	1.93	10.50	1270
13×38	515	494	2682	2,212,500	184,380	1,548,750	129,060	9.19	4-1$\frac{1}{2}$ "	1.93	10.50	1310
13×40	541	520	3000	2,475,000	206,250	1,732,500	144,380	9.72	4-1$\frac{9}{16}$"	2.00	10.54	1380
13×42	569	546	3310	2,730,000	227,500	1,911,000	159,250	10.16	4-1$\frac{5}{8}$ "	2.10	10.56	1440
13×44	596	572	3672	3,010,000	250,830	2,107,000	175,580	10.46	3-1$\frac{7}{8}$ "	1.98	13.82	1135
13×46	622	598	3820	3,150,000	262,500	2,205,000	183,750	11.06	3-1$\frac{15}{16}$"	1.90	13.90	1195
13×48	650	624	4174	3,440,000	287,000	2,408,000	200,900	11.47	3-1$\frac{15}{16}$"	2.00	13.90	1280
13×50	677	650	4575	3,770,000	314,170	2,639,000	219,920	11.98	3-2 "	2.00	14.00	1285
13×52	704	676	4928	4,062,500	338,540	2,843,750	236,980	12.39	3-2$\frac{1}{16}$"	2.10	14.06	1320
14×28	408	392	1550	1,277,500	106,460	894,250	74,520	7.62	5-1$\frac{1}{4}$ "	1.82	9.68	1180
14×30	437.5	420	1785	1,472,500	122,710	1,030,750	85,900	8.20	5-1$\frac{5}{16}$"	2.05	9.73	1265
14×32	467.5	448	1922	1,680,000	140,000	1,176,000	98,000	8.68	5-1$\frac{3}{8}$ "	2.10	9.79	1335
14×34	495	476	2316	1,910,000	159,170	1,337,000	111,420	8.97	4-1$\frac{1}{2}$ "	1.80	9.79	1375
14×36	525	504	2600	2,145,000	178,750	1,501,500	125,130	9.48	4-1$\frac{9}{16}$"	1.90	12.12	1170
14×38	555	532	2880	2,375,000	197,920	1,662,500	138,540	9.89	4-1$\frac{5}{8}$ "	2.00	12.19	1220

TABLE I. — *Beams and Girders.* — Continued.

1	2	3	4	5	6	7	8	9	10	11	12	13
Size of beam or girder.	Weight per lineal foot.	Area of section.	Values of ratio $\frac{I}{y}$.	Safe bending moment. Factor of safety = 3.5.		Safe bending moment. Factor of safety = 5.0.		Area of steel in tension side.	Number and size of rods.	Location of neutral axis below central axis.	Area of concrete in tensile layer.	Tensile stress in concrete in tensile layer.
Inches.	Lbs.	Sq.in.		In. lbs.	Ft. lbs.	In. lbs.	Ft. lbs.	Sq. in.		Inches.	Sq. in.	Lbs. sq. in.
14×40	583	560	3188	2,628,750	219,070	1,840,120	153,340	10.47	4–1 $\frac{5}{8}$″	2.00	12.19	1290
14×42	613	588	3500	2,882,500	240,220	2,017,740	168,140	10.93	4–1 $\frac{11}{16}$″	2.05	12.20	1340
14×44	642	616	3800	3,136,200	261,370	2,195,360	182,940	11.42	4–1 $\frac{11}{16}$″	2.10	12.20	1400
14×46	670	644	4115	3,390,000	282,500	2,373,000	197,750	11.77	3–2 ″	1.80	16.00	1105
14×48	700	672	4500	3,705,000	308,750	2,593,500	216,130	12.36	3–2 $\frac{1}{16}$″	2.00	16.12	1150
14×50	730	700	4910	4,050,000	337,500	2,835,000	236,250	12.85	3–2 $\frac{1}{16}$″	2.00	16.12	1200
14×52	758	728	5305	4,375,000	364,580	3,062,500	255,210	13.27	3–2 $\frac{1}{8}$″	2.50	16.20	1230
14×54	787	756	5748	4,737,500	394,790	3,316,250	276,350	13.72	3–2 $\frac{1}{8}$″	2.00	16.20	1270
14×56	817.5	785	6200	5,110,000	425,830	3,577,000	298,080	14.20	3–2 $\frac{3}{16}$″	2.00	16.27	1310
15×30	468	450	1918	1,580,000	131,800	1,106,000	92,180	8.57	4–1$\frac{1}{2}$ ″	1.70	13.50	950
15×32	500	480	2165	1,785,000	148,750	1,249,500	104,130	8.97	4–1$\frac{1}{2}$ ″	1.70	13.50	1000
15×34	530	510	2488	2,047,500	170,630	1,433,250	119,440	9.62	4–1 $\frac{9}{16}$″	1.80	13.66	1080
15×36	562.5	540	2790	2,300,000	191,670	1,610,000	134,170	10.10	4–1$\frac{5}{8}$ ″	1.80	13.81	1095
15×38	594.5	570	3095	2,550,000	212,500	1,785,000	148,750	10.54	4–1$\frac{5}{8}$ ″	1.85	13.81	1145

15×40	625	600	3320	2,818,750	234,900	1,973,100	164,430	11.19	4-1$\frac{11}{16}$"	1.96	13.86	1210
15×42	656	630	3745	3,087,500	257,300	2,161,260	180,110	11.66	4-1$\frac{11}{16}$"	2.00	13.86	1260
15×44	687.5	660	4070	3,356,250	279,700	2,349,390	195,790	12.18	4-1$\frac{3}{4}$ "	2.03	14.00	1305
15×46	718	690	4400	3,625,000	302,080	2,537,500	211,460	12.75	4-1$\frac{13}{16}$"	2.05	14.07	1360
15×48	750	720	4820	3,970,000	330,830	2,779,000	231,580	13.27	4-1$\frac{13}{16}$"	2.05	14.07	1415
15×50	782	750	5260	4,337,500	361,460	3,036,250	253,020	13.84	3-2$\frac{1}{8}$ "	1.85	18.32	1135
15×52	811	780	5675	4,675,000	389,580	3,272,500	272,710	14.23	3-2$\frac{3}{16}$"	2.06	18.46	1155
15×54	843	810	6170	5,080,000	423,330	3,556,000	296,330	14.88	3-2$\frac{1}{4}$ "	2.20	18.54	1205
15×56	877	840	6640	5,475,000	456,250	3,832,500	319,380	15.37	3-2$\frac{1}{4}$ "	2.22	18.54	1245
15×58	905	870	7150	5,900,000	491,660	4,130,000	344,180	15.80	3-2$\frac{5}{16}$"	2.05	18.60	1275
15×60	937.5	900	7640	6,300,000	525,000	4,410,000	367,500	16.23	3-2$\frac{3}{8}$ "	2.10	18.72	1300
16×32	534	512	2334	1,923,000	160,250	1,346,100	112,180	9.75	5-1$\frac{7}{16}$"	1.86	12.41	1180
16×34	566	544	2645	2,180,000	181,670	1,526,000	127,170	9.84	5-1$\frac{7}{16}$"	1.90	12.41	1190
16×36	600	576	2980	2,457,500	204,790	1,720,250	143,350	10.91	5-1$\frac{1}{2}$ "	2.00	12.75	1280
16×38	634	608	3300	2,725,000	227,080	1,907,500	158,960	11.35	5-1$\frac{1}{2}$ "	2.00	12.75	1335
16×40	666	640	3653	3,012,500	251,040	2,108,750	175,730	11.86	4-1$\frac{3}{4}$ "	1.85	15.75	1130
16×42	700	672	4000	3,300,000	275,000	2,310,000	192,500	12.38	4-1$\frac{3}{4}$ "	1.87	15.75	1180
16×44	733	704	4100	3,582,500	298,960	2,497,750	209,270	12.97	4-1$\frac{13}{16}$"	1.94	15.89	1225
16×46	766	736	4700	3,875,000	322,920	2,712,500	226,040	13.62	4-1$\frac{7}{8}$ "	2.00	15.95	1280
16×48	800	768	5135	4,235,000	352,920	2,964,500	247,040	14.14	4-1$\frac{7}{8}$ "	2.60	15.95	1330
16×50	833	800	5620	4,632,500	386,040	3,242,750	270,230	14.65	3-2$\frac{1}{4}$ "	1.90	20.79	1060
16×52	867	832	6070	5,000,000	416,670	3,500,000	291,660	15.13	3-2$\frac{1}{4}$ "	1.90	20.79	1090
16×54	900	864	6572	5,415,000	451,250	3,790,500	315,880	15.65	3-2$\frac{5}{16}$"	1.90	20.90	1125
16×56	933	896	7090	5,850,000	487,500	4,095,000	341,250	16.25	3-2$\frac{3}{8}$ "	2.00	21.09	1155

TABLE I. — *Beams and Girders.* — Continued.

1	2	3	4	5	6	7	8	9	10	11	12	13
Size of beam or girder.	Weight per lineal foot.	Area of section.	Values of ratio $\frac{I}{y}$.	Safe bending moment. Factor of safety = 3.5.		Safe bending moment. Factor of safety = 5.0.		Area of steel in tension side.	Number and size of rods.	Location of neutral axis below central axis.	Area of concrete in tensile layer.	Tensile stress in concrete in tensile layer.
Inches.	Lbs.	Sq. in.		In. lbs.	Ft. lbs.	In. lbs.	Ft. lbs.	Sq. in.		Inches.	Sq. in.	Lbs. sq. in.
16×58	967	928	7620	6,285,000	523,750	4,399,500	366,630	16.76	3–2$\frac{3}{8}$″	2.00	21.09	1190
16×60	1000	960	8160	6,725,000	560,420	4,707,500	392,290	17.34	3–2$\frac{7}{16}$″	2.10	21.20	1230
16×62	1033	992	8730	7,200,000	600,000	5,040,000	420,000	17.80	3–2$\frac{7}{16}$″	2.00	21.20	1260
16×64	1067	1024	9325	7,685,000	640,420	5,379,500	448,290	18.36	3–2$\frac{1}{2}$″	2.10	21.25	1300
17×34	600	578	2810	2,315,000	192,920	1,620,500	135,040	10.90	5–1$\frac{1}{2}$″	1.85	14.25	1150
17×36	637.5	612	3160	2,605,000	217,080	1,823,500	151,960	11.40	5–1$\frac{1}{2}$″	1.90	14.25	1200
17×38	673	646	3510	2,895,000	241,250	2,026,500	168,880	12.04	5–1$\frac{9}{16}$″	1.96	14.35	1330
17×40	710	680	3920	3,237,500	269,480	2,266,250	188,850	12.70	5–1$\frac{5}{8}$″	2.00	14.42	1325
17×42	743	714	4340	3,575,000	298,000	2,502,500	208,540	13.22	5–1$\frac{5}{8}$″	2.00	14.42	1375
17×44	779	748	4755	3,925,000	327,080	2,747,500	228,960	13.86	5–1$\frac{11}{16}$″	2.10	14.44	1440
17×46	812.5	782	4980	4,110,000	342,500	2,877,000	239,750	14.40	4–1$\frac{15}{16}$″	1.90	17.93	1210
17×48	850	816	5455	4,500,000	375,000	3,150,000	262,500	15.00	4–1$\frac{15}{16}$″	2.00	17.93	1255
17×50	885	850	5970	4,920,000	410,000	3,444,000	287,000	15.60	4–2″	2.00	18.00	1300
17×52	920	884	6450	5,315,000	442,930	3,720,500	310,040	16.11	4–2″	2.10	18.00	1345

17×54	955	928	7,000	5,760,000	480,000	4,032,000	336,000	16.85	4–2 1/16″	2.20	18.08	1390
17×56	990	962	7,530	6,210,000	517,500	4,347,000	362,250	17.20	3–2 7/16″	1.95	23.62	1080
17×58	1025	996	8,110	6,685,000	559,080	4,679,500	391,360	17.74	3–2 7/16″	1.92	23.62	1110
17×60	1060	1030	8,675	7,150,000	595,830	5,005,000	417,080	18.30	3–2 1/2 ″	1.94	23.75	1150
17×62	1095	1064	9,280	7,650,000	637,500	5,355,000	446,250	18.88	3–2 1/2 ″	2.00	23.75	1185
17×64	1130	1098	9,900	8,160,000	680,000	5,712,000	476,000	19.50	3–2 9/16″	2.04	23.90	1225
17×66	1165	1132	10,560	8,710,000	725,830	6,097,000	508,080	20.10	3–2 5/8 ″	2.10	24.00	1260
17×68	1200	1166	11,210	9,250,000	770,830	6,475,000	539,580	20.67	3–2 5/8 ″	2.10	24.00	1295
18×36	675	648	3,355	2,765,000	230,420	1,935,500	161,210	12.10	5–1 1/2 ″	1.74	15.75	1150
18×38	712.5	684	3,715	3,065,000	255,420	2,145,500	178,790	12.64	5–1 5/8 ″	1.90	16.04	1180
18×40	750	720	4,150	3,425,000	285,420	2,397,500	199,790	13.42	5–1 11/16″	1.96	16.10	1250
18×42	787.5	756	4,585	3,782,500	315,210	2,647,750	220,650	14.04	5–1 11/16″	2.00	16.10	1310
18×44	825	792	5,050	4,170,000	347,500	2,919,000	243,250	14.53	4–1 15/16″	1.87	19.76	1110
18×46	862.5	828	5,290	4,355,000	362,920	3,048,500	254,040	15.18	4–1 15/16″	1.90	19.76	1150
18×48	900	864	5,775	4,760,000	396,670	3,332,000	277,670	15.80	4–2 ″	1.92	20.00	1185
18×50	937.5	900	6,300	5,200,000	433,330	3,640,000	303,330	16.48	4–2 1/16″	1.95	20.14	1230
18×52	975	936	6,825	5,625,000	468,750	3,937,500	328,130	17.12	4–2 1/8 ″	2.08	20.23	1270
18×54	1012.5	972	7,400	6,100,000	508,330	4,270,000	355,830	17.75	4–2 1/8 ″	2.10	20.23	1315
18×56	1050	1008	7,980	6,575,000	547,920	4,602,500	383,540	18.36	4–2 1/8 ″	2.10	20.23	1360
18×58	1087.5	1044	3,590	7,075,000	589,580	4,952,500	412,710	19.00	4–2 3/16″	2.12	20.23	1410
18×60	1125	1080	9,180	7,565,000	630,420	5,295,500	441,290	19.36	3–2 9/16″	1.90	26.46	1100
18×62	1162.5	1116	9,825	8,100,000	675,000	5,670,000	472,500	19.94	3–2 5/8 ″	1.93	26.62	1120
18×64	1200	1152	10,480	8,640,000	720,000	3,048,000	504,000	20.52	3–2 5/8 ″	1.95	26.62	1160
18×66	1237.5	1188	11,200	9,230,000	769,170	6,461,000	538,420	21.25	3–2 11/16″	2.00	26.74	1190
18×68	1275	1224	11,870	9,785,000	815,420	6,849,500	570,790	21.84	3–2 11/16″	2.00	26.74	1225

TABLE I. — *Beams and Girders.* — Continued.

1	2	3	4	5	6	7	8	9	10	11	12	13
Size of beam or girder.	Weight per lineal foot.	Area of section.	Values of ratio $\frac{I}{y}$.	Safe bending moment. Factor of safety = 3.5.		Safe bending moment. Factor of safety = 5.0.		Area of steel in tension side.	Number and size of rods.	Location of neutral axis below central axis.	Area of concrete in tensile layer.	Tensile stress in concrete in tensile layer.
Inches.	Lbs.	Sq. in.		In. lbs.	Ft. lbs.	In. lbs.	Ft. lbs.	Sq. in.		Inches.	Sq. in.	Lbs. sq. in.
18×70	1312.5	1260	12,670	10,445,000	870,420	7,311,500	609,290	22.50	3–2$\frac{3}{4}$″	2.00	26.80	1260
18×72	1350	1296	13,700	11,040,000	920,000	7,728,000	644,000	23.05	3–2$\frac{13}{16}$″	2.10	26.90	1290
19×38	752.5	722	3,920	3,230,000	269,166	2,261,000	188,420	13.37	5–1$\frac{11}{16}$″	1.88	17.79	1150
19×40	790	760	4,385	3,617,500	301,460	2,532,250	211,020	14.10	5–1$\frac{11}{16}$″	1.90	17.79	1190
19×42	830	798	4,830	3,985,000	332,080	2,789,500	232,460	14.78	5–1$\frac{3}{4}$″	2.00	17.96	1235
19×44	872	836	5,325	4,395,000	367,080	3,076,500	256,380	15.43	5–1$\frac{3}{4}$″	2.00	17.96	1290
19×46	910	874	5,575	4,597,500	383,130	3,218,250	268,170	16.20	5–1$\frac{13}{16}$″	2.08	18.00	1350
19×48	950	912	6,100	5,035,000	419,580	3,524,500	293,710	16.77	4–2$\frac{1}{16}$″	1.95	22.20	1130
19×50	990	950	6,670	5,495,000	457,920	3,846,500	320,540	17.42	4–2$\frac{1}{8}$″	1.96	22.35	1170
19×52	1030	988	7,200	5,940,000	495,000	4,158,000	346,500	18.00	4–2$\frac{1}{8}$″	2.00	22.35	1210
19×54	1070	1026	7,810	6,435,000	536,250	4,504,500	375,380	18.73	4–2$\frac{3}{16}$″	2.05	22.39	1255
19×56	1110	1064	8,400	6,930,000	577,500	4,851,000	404,250	19.32	4–2$\frac{1}{4}$″	2.08	22.50	1290
19×58	1150	1102	9.055	7,470,000	622,500	5,229,000	435,750	20.05	4–2$\frac{1}{4}$″	2.10	22.50	1340

19×60	1190	1140	9,700	7,990,000	665,830	5,593,000	466,080	20.58	4-2 5/16"	2.10	22.55	1370
19×62	1230	1178	10,390	8,560,000	713,330	5,992,000	499,330	21.10	3-2 11/16"	1.90	29.43	1170
19×64	1270	1216	11,080	9,130,000	760,830	6,391,000	532,580	21.75	3-2 11/16"	2.00	29.43	1110
19×66	1310	1254	11,800	9,740,000	811,670	6,818,000	568,170	22.40	3-2 3/4"	2.00	29.55	1140
19×68	1350	1292	12,560	10,350,000	862,500	7,245,000	603,750	23.00	3-2 13/16"	2.00	29.72	1160
19×70	1390	1330	13,340	11,000,000	916,670	7,700,000	641,670	23.68	3-2 13/16"	2.00	29.72	1200
19×72	1430	1368	14,140	11,650,000	970,830	8,155,000	679,580	24.35	3-2 7/8"	2.08	29.80	1225
19×74	1470	1406	14,970	12,340,000	1,028,330	8,638,000	719,830	25.00	3-2 15/16"	2.08	29.90	1255
19×76	1510	1444	15,770	13,000,000	1,083,330	9,100,000	758,330	25.68	3-2 15/16"	2.20	29.90	1290
20×40	832	800	4,620	3,810,000	317,500	2,667,000	222,250	14.80	5-1 3/4"	1.84	19.71	1125
20×42	875	840	5,095	4,200,000	350,000	2,940,000	245,000	15.46	5-1 3/4"	1.87	19.71	1175
20×44	917.5	880	5,620	4,627,500	385,630	3,239,250	269,940	16.20	5-1 13/16"	1.92	19.75	1230
20×46	957	920	5,870	4,837,500	403,130	3,386,250	282,190	16.94	5-1 7/8"	1.93	19.90	1275
20×48	1000	960	6,425	5,300,000	441,670	3,710,000	309,170	17.72	5-1 15/16"	2.08	20.00	1335
20×50	1042	1000	7,015	5,785,000	482,080	4,049,500	337,460	18.96	5-1 15/16"	2.10	20.00	1420
20×52	1084	1040	7,590	6,250,000	520,830	4,375,000	364,580	19.02	5-1 15/16"	2.13	20.00	1420
20×54	1125	1080	8,340	6,875,000	572,920	4,812,500	401,040	19.71	4-2 1/4"	2.10	24.75	1200
20×56	1167	1120	8,850	7,300,000	608,330	5,110,000	425,830	20.26	4-2 1/4"	1.95	24.75	1240
20×58	1210	1160	9,530	7,860,000	655,000	5,502,000	458,500	21.00	4-2 5/16"	2.03	24.86	1275
20×60	1250	1200	10,180	8,400,000	700,000	5,880,000	490,000	21.60	4-2 3/8"	2.10	25.00	1295
20×62	1292	1240	10,920	9,000,000	750,000	6,300,000	525,000	22.32	4-2 3/8"	2.13	25.00	1340
20×64	1330	1280	11,570	9,630,000	802,500	6,741,000	561,750	23.03	4-2 7/16"	2.13	25.00	1380
20×66	1371	1320	12,440	10,250,000	854,170	7,175,000	597,920	23.71	4-2 7/16"	2.20	25.00	1420
20×68	1415	1360	13,200	10,875,000	906,250	7,612,500	634,380	24.15	3-2 7/8"	1.95	32.67	1110
20×70	1458	1400	14,070	11,600,000	966,670	8,120,000	676,670	24.95	3-2 15/16"	2.00	32.84	1140

TABLE I. — *Beams and Girders.* — Continued.

1	2	3	4	5	6	7	8	9	10	11	12	13
Size of beam or girder.	Weight per lineal foot.	Area of section.	Values of ratio $\frac{I}{y}$.	Safe bending moment. Factor of safety = 3.5.		Safe bending moment. Factor of safety = 5.0.		Area of steel in tension side.	Number and size of rods.	Location of neutral axis below central axis.	Area of concrete in tensile layer.	Tensile stress in concrete in tensile layer.
Inches.	Lbs.	Sq. in.		In. lb.	Ft. lbs.	In. lbs.	Ft. lbs.	Sq. in.		Inches.	Sq. in.	Lbs. sq. in.
20×72	1500	1440	14,860	12,250,000	1,020,830	8,575,000	714,580	25.60	3–2 $\frac{15}{16}$″	2.05	32.84	1170
20×74	1540	1480	15,750	12,975,000	1,081,250	9,082,500	756,880	26.22	3–3 ″	2.00	33.00	1190
20×76	1585	1520	16,500	13,700,000	1,141,670	9,590,000	799,170	26.86	3–3 ″	2.00	33.00	1220
20×78	1625	1560	17,530	14,460,000	1,205,000	10,122,000	843,500	27.66	3–3 $\frac{1}{16}$″	2.10	33.10	1260
20×80	1665	1600	18,500	15,240,000	1,270,000	10,668,000	889,000	28.37	3–3$\frac{1}{8}$ ″	2.08	33.25	1280

NOTE.

With single spans, fixed at the ends, place a reinforcement in the upper side of the beam or girder forming a cantilever from the fixed ends extending toward the center. The total area of this reinforcement should be 66.7 per cent of that placed in the lower side as called for in Table I. The distribution of this reinforcement should be as stated in the description of Tables I*a* and I*b* for the reinforcement in the upper side of continuous girders.

When figuring girders of more than one span, having fixed ends, make the following changes in the amount of reinforcement just given.

With 2 spans decrease the amount by 44.6 per cent.

With 3 spans decrease the amount by 36.0 per cent.

With 4 spans decrease the amount by 38.3 per cent.

With 5 spans decrease the amount by 38.0 per cent.

With 6, 7, 8, and 9 spans, decrease the amount by 38.0 per cent.

DESCRIPTION OF TABLES I*a* AND I*b*.

The following tables are inserted to allow for the effect of continuity of beams or girders over one or more supports, and give the proper steel sections to withstand the bending moments given when produced by a uniformly distributed loading

over equal spans. When these conditions do not exist, it remains for the designer to ascertain the bending moments in the different spans, ignoring the effect of continuity, as this is cared for in the results, and to fix upon the size of concrete girder that will care for the largest moment thus found by referring to the table. Opposite the size just determined will be found the reinforcement to adopt for different parts of the girder. Table I*a* is worked out for two spans. Likewise Table I*b* is for three spans only, but is applicable to any number with a maximum error of 1 per cent for moments over supports, and that on the safe side. For moments within the spans, the following changes should be allowed in the intermediate spans, the outside spans remaining as in the table.

With 4 spans increase the reinforcement in the intermediate spans by 46 per cent.

With 5 spans increase the reinforcement in the intermediate spans by 85 per cent.

With 6 spans increase the reinforcement in the intermediate spans by 74 per cent.

With 7 spans increase the reinforcement in the intermediate spans by 76 per cent.

With 8 spans increase the reinforcement in the intermediate spans by 74 per cent.

With 9 spans increase the reinforcement in the intermediate spans by 74 per cent.

In distributing the reinforcement given in the tables for continuous girders, the following plan may be suggested:

All rods in the underside of the girder should extend from support to support; .3 square inch of steel per square foot section of girder in the upper side of the girder should extend from support to support and lap sufficiently to develop the elastic limit of the section by exposing a sufficient surface to adhesion between the steel and the concrete; one-third of the remaining section in the upper side of the girder, should be one-half the length of the span and center over the support. Another third of the remaining section in the upper side of the girder, should be one-third the length of the span and center over the support. The last third of the remaining section in the upper side of the girder, should be one-fourth the length of the span and center over the support.

NOTE. — The purpose of the continuity of rods in the upper surface is to care for tension caused by an increase of temperature.

The object sought in preparing these tables was to free the designer of the tedious routine in determining the bending moments in the different parts of continuous girders. All spans of the continuous girder should be treated as single spans supported at the ends to determine the maximum moment from which the girder size should be ascertained by reference to the table. It may be unnecessary to state that it is expected that the same size of girder section will be used throughout the length of the continuous girder as determined by the maximum bending moment in the different spans of the girder.

TABLE I*a*.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
2.5×6	7,800	650	.47	1–$\frac{11}{16}$″	.27	1–$\frac{9}{16}$″
2.5×8	15,350	1,280	.52	1–$\frac{3}{4}$″	.29	1–$\frac{9}{16}$″
2.5×10	25,250	2,100	.65	1–$\frac{13}{16}$″	.37	1–$\frac{5}{8}$″
2.5×12	37,800	3,150	.66	1–$\frac{13}{16}$″	.43	1–$\frac{11}{16}$″
3×6	9,380	780	.48	1–$\frac{3}{4}$″	.27	1–$\frac{9}{16}$″
3×8	18,450	1,540	.59	1–$\frac{13}{16}$″	.33	1–$\frac{5}{8}$″
3×10	30,000	2,500	.66	1–$\frac{13}{16}$″	.37	1–$\frac{5}{8}$″
3×12	45,500	3,790	.78	1–$\frac{7}{8}$″	.44	1–$\frac{11}{16}$″
3×14	63,500	5,290	.88	1–$\frac{15}{16}$″	.50	1–$\frac{3}{4}$″
4×8	24,500	2,040	.75	1–$\frac{7}{8}$″	.42	1–$\frac{11}{16}$″
4×10	40,500	3,380	.86	1–$\frac{15}{16}$″	.49	1–$\frac{3}{4}$″
4×12	60,500	5,040	.99	1–1″	.56	1–$\frac{3}{4}$″
4×14	84,500	7,040	1.14	1–1$\frac{1}{16}$″	.64	1–$\frac{13}{16}$″
4×16	112,500	9,380	1.26	1–1$\frac{1}{8}$″	.71	1–$\frac{7}{8}$″
5×10	50,650	4,220	1.15	2–$\frac{3}{4}$″	.65	1–$\frac{13}{16}$″
5×12	75,600	6,300	1.32	2–$\frac{13}{16}$″	.74	1–$\frac{7}{8}$″
5×14	105,500	8,790	1.46	2–$\frac{7}{8}$″	.82	1–$\frac{15}{16}$″
5×16	140,500	11,710	1.68	2–$\frac{15}{16}$″	.95	1–1″
5×18	180,000	15,000	1.86	2–1″	1.05	1–1$\frac{1}{16}$″
5×20	225,500	18,790	2.06	2–1$\frac{1}{16}$″	1.17	1–1$\frac{1}{8}$″
6×12	91,000	7,580	1.53	2–$\frac{7}{8}$″	.86	1–$\frac{15}{16}$″
6×14	127,000	10,580	1.78	2–$\frac{15}{16}$″	1.00	1–1″
6×16	168,500	14,040	1.97	2–1″	1.12	1–1$\frac{1}{16}$″
6×18	217,000	18,080	2.19	2–1$\frac{1}{16}$″	1.24	1–1$\frac{1}{8}$″
6×20	270,500	22,540	2.40	2–1$\frac{1}{8}$″	1.36	2–$\frac{7}{8}$″
6×22	332,500	27,710	2.62	2–1$\frac{1}{8}$″	1.48	2–$\frac{7}{8}$″
6×24	398,500	33,210	2.84	2–1$\frac{1}{4}$″	1.61	2–$\frac{15}{16}$″

TABLE I*a*. — Continued.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
7×14	148,000	12,330	2.14	3–$\frac{7}{8}''$	1.21	1–$1\frac{1}{8}''$
7×16	197,000	16,420	2.41	3–$\frac{15}{16}''$	1.36	2–$\frac{7}{8}''$
7×18	252,500	21,040	2.62	3–$\frac{15}{16}''$	1.48	2–$\frac{7}{8}''$
7×20	315,000	26,250	2.87	3–1″	1.62	2–$\frac{15}{16}''$
7×22	387,500	32,280	3.11	3–$1\frac{1}{16}''$	1.76	2–$\frac{15}{16}''$
7×24	464,500	38,710	3.27	3–$1\frac{1}{16}''$	1.85	2–1″
7×26	547,500	45,630	3.53	3–$1\frac{1}{16}''$	2.00	2–1″
7×28	639,500	53,290	3.77	3–$1\frac{1}{8}''$	2.14	2–$1\frac{1}{16}''$
8×16	225,000	18,750	2.63	3–$\frac{15}{16}''$	1.49	2–$\frac{7}{8}''$
8×18	288,500	24,040	2.90	3–1″	1.64	2–$\frac{15}{16}''$
8×20	360,500	30,040	3.24	3–$1\frac{1}{16}''$	1.85	2–1″
8×22	442,500	36,880	3.55	3–$1\frac{1}{8}''$	2.01	2–1″
8×24	531,000	44,250	3.82	3–$1\frac{1}{8}''$	2.16	2–$1\frac{1}{16}''$
8×26	625,000	52,080	4.09	3–$1\frac{3}{16}''$	2.32	2–$1\frac{1}{8}''$
8×28	725,000	60,420	4.32	3–$1\frac{3}{16}''$	2.44	2–$1\frac{1}{8}''$
8×30	840,000	70,000	4.65	3–$1\frac{1}{4}''$	2.63	2–$1\frac{3}{16}''$
8×32	962,000	80,170	4.97	3–$1\frac{5}{16}''$	2.81	2–$1\frac{3}{16}''$
9×18	324,000	27,000	3.38	4–$\frac{15}{16}''$	1.91	2–1″
9×20	406,000	33,830	3.73	4–1″	2.11	2–$1\frac{1}{16}''$
9×22	498,500	41,540	4.03	4–1″	2.28	2–$1\frac{1}{8}''$
9×24	597,000	49,750	4.37	4–$1\frac{1}{16}''$	2.47	2–$1\frac{1}{8}''$
9×26	702,500	58,540	4.65	4–$1\frac{1}{16}''$	2.63	3–$\frac{15}{16}''$
9×28	800,000	66,660	4.73	3–$1\frac{1}{4}''$	2.68	3–1″
9×30	946,500	78,870	5.22	3–$1\frac{3}{8}''$	2.94	3–1″
9×32	1,082,500	90,210	5.54	3–$1\frac{3}{8}''$	3.13	3–$1\frac{1}{16}''$
9×34	1,227,500	102,290	5.87	3–$1\frac{7}{16}''$	3.31	3–$1\frac{1}{16}''$
9×36	1,380,000	115,000	6.16	3–$1\frac{7}{16}''$	3.48	3–$1\frac{1}{8}''$
10×20	451,000	37,580	4.10	4–1″	2.32	3–$\frac{7}{8}''$
10×22	554,000	46,170	4.43	4–$1\frac{1}{16}''$	2.51	3–$\frac{15}{16}''$

TABLE I*a*. — Continued.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
10 × 24	664,000	55,330	4.90	4–1 $\frac{1}{8}$″	2.77	3–1″
10 × 26	780,000	65,000	5.12	4–1 $\frac{1}{8}$″	2.89	3–1″
10 × 28	914,000	76,170	5.40	3–1 $\frac{3}{8}$″	3.05	3–1 $\frac{1}{16}$″
10 × 30	1,050,000	87,500	5.73	3–1 $\frac{7}{16}$″	3.24	3–1 $\frac{1}{16}$″
10 × 32	1,200,000	100,000	6.09	3–1 $\frac{1}{2}$″	3.44	3–1 $\frac{1}{8}$″
10 × 34	1,365,000	113,750	6.71	3–1 $\frac{1}{2}$″	3.79	3–1 $\frac{1}{8}$″
10 × 36	1,532,000	127,670	6.80	3–1 $\frac{9}{16}$″	3.85	3–1 $\frac{3}{16}$″
10 × 38	1,702,000	141,830	7.14	3–1 $\frac{9}{16}$″	4.03	3–1 $\frac{3}{16}$″
10 × 40	1,905,000	158,750	7.35	3–1 $\frac{5}{8}$″	4.15	3–1 $\frac{3}{16}$″
11 × 22	610,000	50,830	4.97	5–1″	2.80	3–1″
11 × 24	729,000	60,750	5.36	5–1 $\frac{1}{16}$″	3.03	3–1″
11 × 26	858,000	71,500	5.68	5–1 $\frac{1}{16}$″	3.21	3–1 $\frac{1}{16}$″
11 × 28	1,002,500	83,540	6.02	4–1 $\frac{1}{4}$″	3.40	3–1 $\frac{1}{16}$″
11 × 30	1,157,000	96,420	6.48	4–1 $\frac{1}{4}$″	3.66	3–1 $\frac{1}{8}$″
11 × 32	1,325,000	110,420	6.65	3–1 $\frac{1}{2}$″	3.75	3–1 $\frac{1}{8}$″
11 × 34	1,500,000	125,000	7.07	3–1 $\frac{9}{16}$″	3.96	4–1″
11 × 36	1,685,000	140,420	7.45	3–1 $\frac{5}{8}$″	4.20	4–1 $\frac{1}{16}$″
11 × 38	1,871,000	155,920	7.80	3–1 $\frac{11}{16}$″	4.40	4–1 $\frac{1}{16}$″
11 × 40	2,092,000	174,330	8.24	3–1 $\frac{11}{16}$″	4.65	4–1 $\frac{1}{8}$″
11 × 42	2,312,500	192,710	8.61	3–1 $\frac{3}{4}$″	4.86	4–1 $\frac{1}{8}$″
11 × 44	2,540,000	211,670	8.95	3–1 $\frac{3}{4}$″	5.05	4–1 $\frac{1}{8}$″
12 × 24	796,000	66,330	5.58	5–1 $\frac{1}{16}$″	3.15	3–1 $\frac{1}{16}$″
12 × 26	937,000	78,080	6.08	4–1 $\frac{1}{4}$″	3.43	3–1 $\frac{1}{16}$″
12 × 28	1,095,000	91,250	6.51	4–1 $\frac{5}{16}$″	3.68	3–1 $\frac{1}{8}$″
12 × 30	1,262,500	105,210	6.97	4–1 $\frac{5}{16}$″	3.93	4–1″
12 × 32	1,440,000	120,000	7.33	4–1 $\frac{3}{8}$″	4.14	4–1 $\frac{1}{16}$″
12 × 34	1,640,000	136,670	7.61	4–1 $\frac{7}{16}$″	4.30	4–1 $\frac{1}{16}$″
12 × 36	1,841,000	153,420	8.12	3–1 $\frac{11}{16}$″	4.58	4–1 $\frac{1}{8}$″
12 × 38	2,032,000	169,330	8.42	3–1 $\frac{3}{4}$″	4.75	4–1 $\frac{1}{8}$″
12 × 40	2,249,000	187,410	8.92	3–1 $\frac{3}{4}$″	5.03	5–1″

TABLE I*a*. — Continued.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
12×42	2,466,000	205,490	9.35	3–1 13/16″	5.28	5–1 1/16″
12×44	2,683,000	223,570	9.85	3–1 13/16″	5.56	5–1 1/16″
12×46	2,900,000	241,670	10.25	3–1 7/8″	5.80	5–1 1/8″
12×48	3,175,000	264,580	10.75	3–1 7/8″	6.08	5–1 1/8″
13×26	1,015,000	84,580	6.60	5–1 3/16″	3.73	5–7/8″
13×28	1,187,500	98,960	7.05	5–1 3/16″	3.98	5–15/16″
13×30	1,367,500	115,630	7.68	5–1 1/4″	4.34	5–15/16″
13×32	1,560,000	130,000	8.08	5–1 5/16″	4.57	5–1″
13×34	1,770,000	147,500	8.49	5–1 5/16″	4.80	5–1″
13×36	1,995,000	166,250	8.92	4–1 1/2″	5.04	5–1″
13×38	2,212,500	184,380	9.19	4–1 1/2″	5.20	5–1 1/16″
13×40	2,475,000	206,250	9.72	4–1 9/16″	5.50	5–1 1/16″
13×42	2,730,000	227,500	10.16	4–1 5/8″	5.75	5–1 1/8″
13×44	3,010,000	250,830	10.46	3–1 7/8″	5.91	5–1 1/8″
13×46	3,150,000	262,500	11.06	3–1 15/16″	6.25	5–1 1/8″
13×48	3,440,000	287,000	11.47	3–1 15/16″	6.48	5–1 3/16″
13×50	3,770,000	314,170	11.98	3–2″	6.75	5–1 3/16″
13×52	4,062,500	338,540	12.39	3–2 1/16″	6.98	5–1 3/16″
14×28	1,277,500	106,460	7.62	5–1 1/4″	4.31	5–15/16″
14×30	1,472,500	122,710	8.20	5–1 5/16″	4.63	5–1″
14×32	1,680,000	140,000	8.68	5–1 3/8″	4.91	5–1″
14×34	1,910,000	159,170	8.97	4–1 1/2″	5.07	5–1″
14×36	2,145,000	178,750	9.48	4–1 9/16″	5.36	5–1 1/16″
14×38	2,375,000	197,920	9.89	4–1 5/8″	5.59	5–1 1/16″
14×40	2,628,750	219,070	10.47	4–1 5/8″	5.91	5–1 1/8″
14×42	2,882,500	240,220	10.93	4–1 11/16	6.18	5–1 1/8″
14×44	3,136,200	261,370	11.42	4–1 11/16″	6.46	5–1 3/16″
14×46	3,390,000	282,500	11.77	3–2″	6.65	5–1 3/16″
14×48	3,705,000	308,750	12.36	3–2 1/16″	6.98	5–1 3/16″
14×50	4,050,000	337,500	12.85	3–2 1/16″	7.26	5–1 1/4″

TABLE I*a*. — Continued.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
14×52	4,375,000	364,580	13.27	3–2⅛″	7.50	5–1¼″
14×54	4,737,500	394,790	13.72	3–2⅛″	7.75	5–1¼″
14×56	5,110,000	425,830	14.20	3–2 3/16″	8.02	5–1 5/16″
15×30	1,580,000	131,800	8.57	4–1½″	4.84	5–1″
15×32	1,785,000	148,750	8.97	4–1½″	5.07	5–1 1/16″
15×34	2,047,500	170,630	9.62	4–1 9/16″	5.43	5–1 1/16″
15×36	2,300,000	191,670	10.10	4–1⅝″	5.71	5–1⅛″
15×38	2,550,000	212,500	10.54	4–1⅝″	5.94	5–1⅛″
15×40	2,818,750	234,900	11.19	4–1 11/16″	6.32	5–1⅛″
15×42	3,087,500	257,300	11.66	4–1 11/16″	6.59	5–1 3/16″
15×44	3,356,250	279,700	12.18	4–1¾″	6.88	5–1 3/16″
15×46	3,625,000	302,080	12.75	4–1 13/16″	7.21	5–1¼″
15×48	3,970,000	330,830	13.27	4–1 13/16″	7.50	5–1¼″
15×50	4,337,500	361,460	13.84	3–2⅛″	7.82	5–1¼″
15×52	4,675,000	389,580	14.23	3–2 3/16″	8.04	5–1 5/16″
15×54	5,080,000	423,330	14.88	3–2¼″	8.41	5–1 5/16″
15×56	5,475,000	456,250	15.37	3–2¼″	8.68	5–1⅜″
15×58	5,900,000	491,660	15.80	3–2 5/16″	8.93	5–1⅜″
15×60	6,300,000	525,000	16.23	3–2⅜″	9.17	5–1⅜″
16×32	1,923,000	160,250	9.75	5–1 7/16″	5.51	5–1 1/16″
16×34	2,180,000	181,670	9.84	5–1 7/16″	5.56	5–1 1/16″
16×36	2,457,500	204,790	10.91	5–1½″	6.17	5–1⅛″
16×38	2,725,000	227,080	11.35	5–1½″	6.42	5–1⅛″
16×40	3,012,500	251,040	11.86	4–1¾″	6.70	5–1⅛″
16×42	3,300,000	275,000	12.38	4–1¾″	7.00	5–1 3/16″
16×44	3,582,500	298,960	12.97	4–1 13/16″	7.33	5–1¼″
16×46	3,875,000	322,920	13.62	4–1⅞″	7.70	5–1¼″
16×48	4,235,000	352,920	14.14	4–1⅞″	8.00	5–1 5/16″
16×50	4,632,500	386,040	14.65	3–2¼″	8.28	5–1 5/16″
16×52	5,000,000	416,670	15.13	3–2¼″	8.53	3–1 11/16″

TABLE I*a*. — Continued.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
16×54	5,415,000	451,250	15.65	3–2$\frac{5}{16}$"	8.83	3–1$\frac{11}{16}$"
16×56	5,850,000	487,500	16.25	3–2$\frac{3}{8}$"	9.17	3–1$\frac{3}{4}$"
16×58	6,285,000	523,750	16.76	3–2$\frac{3}{8}$"	9.46	3–1$\frac{13}{16}$"
16×60	6,725,000	560,420	17.34	3–2$\frac{7}{16}$"	9.78	3–1$\frac{13}{16}$"
16×62	7,200,000	600,000	17.80	3–2$\frac{7}{16}$"	10.08	3–1$\frac{7}{8}$"
16×64	7,685,000	640,420	18.36	3–2$\frac{1}{2}$"	10.38	3–1$\frac{7}{8}$"
17×34	2,315,000	192,920	10.90	5–1$\frac{1}{2}$"	6.15	5–1$\frac{1}{8}$"
17×36	2,605,000	217,080	11.40	5–1$\frac{1}{2}$"	6.43	5–1$\frac{3}{16}$"
17×38	2,895,000	241,250	12.04	5–1$\frac{9}{16}$"	6.80	5–1$\frac{3}{16}$"
17×40	3,237,500	269,480	12.70	5–1$\frac{5}{8}$"	7.17	5–1$\frac{1}{4}$"
17×42	3,575,000	298,000	13.22	5–1$\frac{5}{8}$"	7.46	5–1$\frac{1}{4}$"
17×44	3,925,000	327,080	13.86	5–1$\frac{11}{16}$"	7.82	5–1$\frac{1}{4}$"
17×46	4,110,000	342,500	14.40	4–1$\frac{15}{16}$"	8.13	4–1$\frac{7}{16}$"
17×48	4,500,000	375,000	15.00	4–1$\frac{15}{16}$"	8.44	4–1$\frac{1}{2}$"
17×50	4,920,000	410,000	15.60	4–2"	8.77	4–1$\frac{1}{2}$"
17×52	5,315,000	442,930	16.11	4–2"	9.07	4–1$\frac{9}{16}$"
17×54	5,760,000	480,000	16.85	4–2$\frac{1}{16}$"	9.48	4–1$\frac{9}{16}$"
17×56	6,210,000	517,500	17.20	3–2$\frac{7}{16}$"	9.67	3–1$\frac{13}{16}$"
17×58	6,685,000	559,080	17.74	3–2$\frac{7}{16}$"	9.98	3–1$\frac{7}{8}$"
17×60	7,150,000	595,830	18.30	3–2$\frac{1}{2}$"	10.35	3–1$\frac{7}{8}$"
17×62	7,650,000	637,500	18.88	3–2$\frac{1}{2}$"	10.67	3–1$\frac{15}{16}$"
17×64	8,160,000	680,000	19.50	3–2$\frac{9}{16}$"	11.03	3–1$\frac{15}{16}$"
17×66	8,710,000	725,830	20.10	3–2$\frac{5}{8}$"	11.36	3–2"
17×68	9,250,000	770,830	20.67	3–2$\frac{5}{8}$"	11.70	3–2"
18×36	2,765,000	230,420	12.10	5–1$\frac{1}{2}$"	6.84	5–1$\frac{3}{16}$"
18×38	3,065,000	255,420	12.64	5–1$\frac{5}{8}$"	7.13	5–1$\frac{1}{4}$"
18×40	3,425,000	285,420	13.42	5–1$\frac{11}{16}$"	7.57	5–1$\frac{1}{4}$"
18×42	3,782,500	315,210	14.04	5–1$\frac{11}{16}$"	7.93	5–1$\frac{5}{16}$"
18×44	4,170,000	347,500	14.53	4–1$\frac{15}{16}$"	8.20	4–1$\frac{7}{16}$"
18×46	4,355,000	362,920	15.18	4–1$\frac{15}{16}$"	8.56	4–1$\frac{1}{2}$"

TABLE I*a*. — Continued.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
18×48	4,760,000	396,670	15.80	4–2 ″	8.92	4–1½ ″
18×50	5,200,000	433,330	16.48	4–2$\frac{1}{16}$″	9.30	4–1$\frac{9}{16}$″
18×52	5,625,000	468,750	17.12	4–2⅛ ″	9.65	4–1$\frac{9}{16}$″
18×54	6,100,000	508,330	17.75	4–2⅛ ″	10.03	4–1⅝ ″
18×56	6,575,000	547,920	18.36	4–2⅛ ″	10.37	4–1⅝ ″
18×58	7,075,000	589,580	19.00	4–2$\frac{3}{16}$″	10.74	4–1$\frac{11}{16}$″
18×60	7,565,000	630,420	19.36	3–2$\frac{9}{16}$″	10.95	3–1$\frac{15}{16}$″
18×62	8,100,000	675,000	19.94	3–2⅝ ″	11.26	3–1$\frac{15}{16}$″
18×64	8,640,000	720,000	20.52	3–2⅝ ″	11.59	3–2 ″
18×66	9,230,000	769,170	21.25	3–2$\frac{11}{16}$″	12.00	3–2 ″
18×68	9,785,000	815,420	21.84	3–2$\frac{11}{16}$″	12.33	3–2$\frac{1}{16}$″
18×70	10,445,000	870,420	22.50	3–2¾ ″	12.72	3–2$\frac{1}{16}$″
18×72	11,040,000	920,000	23.05	3–2$\frac{13}{16}$″	13.03	3–2⅛″
19×38	3,230,000	269,166	13.37	5–1$\frac{11}{16}$″	7.55	5–1¼ ″
19×40	3,617,500	301,460	14.10	5–1$\frac{11}{16}$″	7.96	5–1$\frac{5}{16}$″
19×42	3,985,000	332,080	14.78	5–1¾ ″	8.35	5–1$\frac{5}{16}$″
19×44	4,395,000	367,080	15.43	5–1¾ ″	8.71	5–1⅜ ″
19×46	4,597,500	383,130	16.20	5–1$\frac{13}{16}$″	9.15	5–1⅜ ″
19×48	5,035,000	419,580	16.77	4–2$\frac{1}{16}$″	9.48	4–1$\frac{9}{16}$″
19×50	5,495,000	457,920	17.42	4–2⅛ ″	9.83	4–1⅝ ″
19×52	5,940,000	495,000	18.00	4–2⅛ ″	10.16	4–1⅝ ″
19×54	6,435,000	536,250	18.73	4–2$\frac{3}{16}$″	10.57	4–1⅝ ″
19×56	6,930,000	577,500	19.32	4–2¼ ″	10.90	4–1$\frac{11}{16}$″
19×58	7,470,000	622,500	20.05	4–2¼ ″	11.32	4–1$\frac{11}{16}$″
19×60	7,990,000	665,830	20.58	4–2$\frac{5}{16}$″	11.61	4–1¾ ″
19×62	8,560,000	713,330	21.10	3–2$\frac{11}{16}$″	11.91	3–2 ″
19×64	9,130,000	760,830	21.75	3–2$\frac{11}{16}$″	12.28	3–2$\frac{1}{16}$″
19×66	9,740,000	811,670	22.40	3–2¾ ″	12.65	3–2$\frac{1}{16}$″
19×68	10,350,000	862,500	23.00	3–2$\frac{13}{16}$″	13.00	3–2⅛ ″
19×70	11,000,000	916,670	23.68	3–2$\frac{13}{16}$″	13.37	3–2⅛ ″
19×72	11,650,000	970,830	24.35	3–2⅞ ″	13.75	3–2$\frac{3}{16}$″

TABLE I*a*. — Continued.

1	2	3	4	5	6	7
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over central support. (In upper side of girder.)		Reinforcement throughout spans. (In lower side of girder.)	
			Area of metal.	No. and size of rods.	Area of metal.	No. and size of rods.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.	
19×74	12,340,000	1,028,330	25.00	3–2$\frac{15}{16}$″	14.10	3–2$\frac{3}{16}$″
19×76	13,000,000	1,083,330	25.68	3–2$\frac{15}{16}$″	14.50	3–2$\frac{1}{4}$″
20×40	3,810,000	317,500	14.80	5–1$\frac{3}{4}$″	8.36	5–1$\frac{5}{16}$″
20×42	4,200,000	350,000	15.46	5–1$\frac{3}{4}$″	8.73	5–1$\frac{3}{8}$″
20×44	4,627,500	385,630	16.20	5–1$\frac{13}{16}$″	9.15	5–1$\frac{3}{8}$″
20×46	4,837,500	403,130	16.94	5–1$\frac{7}{8}$″	9.57	5–1$\frac{7}{16}$″
20×48	5,300,000	441,670	17.72	5–1$\frac{15}{16}$″	10.01	5–1$\frac{7}{16}$″
20×50	5,785,000	482,080	18.96	5–1$\frac{15}{16}$″	10.44	5–1$\frac{1}{2}$″
20×52	6,250,000	520,830	19.02	5–1$\frac{15}{16}$″	10.76	5–1$\frac{1}{2}$″
20×54	6,875,000	572,920	19.71	4–2$\frac{1}{4}$″	11.14	4–1$\frac{11}{16}$″
20×56	7,300,000	608,330	20.26	4–2$\frac{1}{4}$″	11.45	4–1$\frac{3}{4}$″
20×58	7,860,000	655,000	21.00	4–2$\frac{5}{16}$″	11.87	4–1$\frac{3}{4}$″
20×60	8,400,000	700,000	21.60	4–2$\frac{3}{8}$″	12.20	4–1$\frac{3}{4}$″
20×62	9,000,000	750,000	22.32	4–2$\frac{3}{8}$″	12.60	4–1$\frac{13}{16}$″
20×64	9,630,000	802,500	23.03	4–2$\frac{7}{16}$″	13.01	4–1$\frac{13}{16}$″
20×66	10,250,000	854,170	23.71	4–2$\frac{7}{16}$″	13.40	4–1$\frac{7}{8}$″
20×68	10,875,000	906,250	24.15	3–2$\frac{7}{8}$″	13.65	3–2$\frac{3}{16}$″
20×70	11,600,000	966,670	24.95	3–2$\frac{15}{16}$″	14.10	3–2$\frac{3}{16}$″
20×72	12,250,000	10,20,830	25.60	3–2$\frac{15}{16}$″	14.47	3–2$\frac{1}{4}$″
20×74	12,975,000	1,081,250	26.22	3–3″	14.82	3–2$\frac{1}{4}$″
20×76	13,700,000	1,141,670	26.86	3–3″	15.19	3–2$\frac{1}{4}$″
20×78	14,460,000	1,205,000	27.66	3–3$\frac{1}{16}$″	15.64	3–2$\frac{5}{16}$″
20×80	15,240,000	1,270,000	28.37	3–3$\frac{1}{8}$″	16.01	3–2$\frac{5}{16}$″

Key to Using Tables Ia, Ib, I, and II.

In cases of continuous girders with 2 spans, use Table I*a*.

In cases of continuous girders with 3 spans, use Table I*b*.

In cases of continuous girders with 4 or more spans, use Table I*b*, with the modifications given in the description for Tables I*a* and I*b*.

In cases of girders or beams with one span only supported at the ends, when loaded uniformly, and when the span is known, use Table II.

In cases of girders or beams with one span only, supported at the ends and receiving concentrated loads, determine the maximum bending moment, and use Table I.

In cases of girders or beams fixed at one or both ends, use Tables I and II, modified as stated in the note following Table I.

TABLE I*b*.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span. (In lower side of girder.)		Reinforcement, outside spans. (In lower side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
2.5×6	9,700	810	.47	1–$\frac{11}{16}$″	.12	1–$\frac{3}{8}$″	.38	1–$\frac{5}{8}$″
2.5×8	19,100	1,600	.52	1–$\frac{3}{4}$″	.13	1–$\frac{3}{8}$″	.42	1–$\frac{11}{16}$″
2.5×10	31,600	2,620	.65	1–$\frac{13}{16}$″	.16	1–$\frac{7}{16}$″	.52	1–$\frac{3}{4}$″
2.5×12	47,200	3,940	.66	1–$\frac{13}{16}$″	.19	1–$\frac{7}{16}$″	.62	1–$\frac{13}{16}$″
3×6	11,700	975	.48	1–$\frac{3}{4}$″	.12	1–$\frac{3}{8}$″	.38	1–$\frac{5}{8}$″
3×8	23,050	1,930	.59	1–$\frac{13}{16}$″	.15	1–$\frac{7}{16}$″	.47	1–$\frac{11}{16}$″
3×10	37,500	3,120	.66	1–$\frac{13}{16}$″	.17	1–$\frac{7}{16}$″	.53	1–$\frac{3}{4}$″
3×12	56,900	4,740	.78	1–$\frac{7}{8}$″	.19	1–$\frac{7}{16}$″	.62	1–$\frac{13}{16}$″
3×14	79,400	6,510	.88	1–$\frac{15}{16}$″	.22	1–$\frac{1}{2}$″	.70	1–$\frac{7}{8}$″
4×8	30,600	2,550	.75	1–$\frac{7}{8}$″	.19	1–$\frac{7}{16}$″	.60	1–$\frac{13}{16}$″
4×10	50,600	4,220	.86	1–$\frac{15}{16}$″	.21	1–$\frac{1}{2}$″	.69	1–$\frac{7}{8}$″
4×12	75,600	6,300	.99	1–1″	.25	1–$\frac{1}{2}$″	.79	1–$\frac{15}{16}$″
4×14	106,000	8,800	1.14	1–1$\frac{1}{16}$″	.29	1–$\frac{9}{16}$″	.91	1–1″
4×16	140,500	11,740	1.26	1–1$\frac{1}{8}$″	.32	1–$\frac{9}{16}$″	1.01	1–1″
5×10	63,300	5,280	1.15	2–$\frac{3}{4}$″	.29	1–$\frac{9}{16}$″	.91	1–1″
5×12	94,500	7,870	1.32	2–$\frac{13}{16}$″	.33	1–$\frac{5}{8}$″	1.06	1–1$\frac{1}{16}$″
5×14	132,000	11,000	1.46	2–$\frac{7}{8}$″	.37	1–$\frac{5}{8}$″	1.17	1–1$\frac{1}{8}$″
5×16	175,600	14,650	1.68	2–$\frac{15}{16}$″	.42	1–$\frac{11}{16}$″	1.34	1–1$\frac{3}{16}$″
5×18	225,000	18,750	1.86	2–1″	.47	1–$\frac{11}{16}$″	1.49	1–1$\frac{1}{4}$″
5×20	282,000	23,480	2.06	2–1$\frac{1}{16}$″	.52	1–$\frac{3}{4}$″	1.65	1–1$\frac{5}{16}$″
6×12	113,600	9,470	1.55	2–$\frac{7}{8}$″	.38	1–$\frac{5}{8}$″	1.12	1–1$\frac{1}{16}$″
6×14	159,000	13,240	1.78	2–$\frac{15}{16}$″	.45	1–$\frac{11}{16}$″	1.42	2–$\frac{7}{8}$″
6×16	210,400	17,550	1.97	2–1″	.49	1–$\frac{3}{4}$″	1.58	2–$\frac{15}{16}$″
6×18	271,500	22,600	2.19	2–1$\frac{1}{16}$″	.55	1–$\frac{3}{4}$″	1.75	2–$\frac{15}{16}$″
6×20	338,000	28,200	2.40	2–1$\frac{1}{8}$″	.60	1–$\frac{13}{16}$″	1.92	2–1″
6×22	415,500	34,690	2.62	2–1$\frac{1}{8}$″	.65	1–$\frac{13}{16}$″	2.10	2–1$\frac{1}{16}$″
6×24	498,500	41,520	2.84	2–1$\frac{1}{4}$″	.71	1–$\frac{7}{8}$″	2.27	2–1$\frac{1}{16}$″

TABLE I*b*. — Continued.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span (In lower side of girder.)		Reinforcement, outside spans. (In lower side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
7×14	185,000	15,420	2.14	3–$\frac{7}{8}$″	.53	1–$\frac{3}{4}$″	1.71	2–$\frac{15}{16}$″
7×16	246,000	20,550	2.41	3–$\frac{15}{16}$″	.60	1–$\frac{13}{16}$″	1.93	2–1″
7×18	316,000	26,260	2.62	3–$\frac{15}{16}$″	.66	1–$\frac{13}{16}$″	2.10	2–1$\frac{1}{16}$″
7×20	394,000	32,800	2.87	3–1″	.72	1–$\frac{7}{8}$″	2.30	2–1$\frac{1}{8}$″
7×22	485,000	40,400	3.11	3–1$\frac{1}{16}$″	.78	1–$\frac{15}{16}$″	2.49	2–1$\frac{1}{8}$″
7×24	580,000	48,500	3.27	3–1$\frac{1}{16}$″	.82	1–$\frac{15}{16}$″	2.62	2–1$\frac{3}{16}$″
7×26	685,000	57,100	3.53	3–1$\frac{1}{16}$″	.88	1–$\frac{15}{16}$″	2.82	2–1$\frac{3}{16}$″
7×28	800,000	66,700	3.77	3–1$\frac{1}{8}$″	.94	1–1″	3.02	2–1$\frac{1}{4}$″
8×16	281,500	23,480	2.63	3–$\frac{15}{16}$″	.66	1–$\frac{13}{16}$″	2.10	2–1$\frac{1}{16}$″
8×18	360,600	30,100	2.90	3–1″	.73	1–$\frac{7}{8}$″	2.32	2–1$\frac{1}{8}$″
8×20	450,500	37,600	3.24	3–1$\frac{1}{16}$″	.81	1–$\frac{15}{16}$″	2.59	3–$\frac{15}{16}$″
8×22	553,000	46,150	3.55	3–1$\frac{1}{8}$″	.89	1–1″	2.84	3–1″
8×24	664,000	55,400	3.82	3–1$\frac{1}{8}$″	.96	1–1″	3.06	3–1″
8×26	781,000	65,200	4.09	3–1$\frac{3}{16}$	1.02	1–1$\frac{1}{16}$″	3.27	3–1$\frac{1}{16}$″
8×28	906,000	75,550	4.32	3–1$\frac{3}{16}$″	1.08	1–1$\frac{1}{16}$″	3.46	3–1$\frac{1}{8}$″
8×30	1,050,000	87,600	4.65	3–1$\frac{1}{4}$″	1.14	1–1$\frac{1}{8}$″	3.72	3–1$\frac{1}{8}$″
8×32	1,202,000	100,800	4.97	3–1$\frac{5}{16}$″	1.24	1–1$\frac{1}{8}$″	3.98	3–1$\frac{3}{16}$″
9×18	405,000	33,750	3.38	4–$\frac{15}{16}$″	.85	1–$\frac{15}{16}$″	2.70	3–1″
9×20	507,000	42,250	3.73	4–1″	.93	1–1″	2.93	3–1″
9×22	622,000	52,000	4.03	4–1″	1.01	1–1″	3.21	3–1$\frac{1}{16}$″
9×24	745,000	62,150	4.37	4–1$\frac{1}{16}$″	1.09	1–1$\frac{1}{16}$″	3.50	3–1$\frac{1}{8}$″
9×26	877,000	73,150	4.65	4–1$\frac{1}{16}$″	1.16	1–1$\frac{1}{8}$″	3.72	3–1$\frac{1}{8}$″
9×28	1,000,000	83,300	4.73	3–1$\frac{1}{4}$″	1.18	2–$\frac{13}{16}$″	3.78	3–1$\frac{1}{8}$″
9×30	1,185,000	98,500	5.22	3–1$\frac{3}{8}$″	1.31	2–$\frac{13}{16}$″	4.17	4–1$\frac{1}{16}$″
9×32	1,353,000	112,800	5.54	3–1$\frac{3}{8}$″	1.39	2–$\frac{7}{8}$″	4.43	4–1$\frac{1}{16}$″
9×34	1,535,000	128,000	5.87	3–1$\frac{7}{13}$	1.47	2–$\frac{7}{8}$″	4.70	4–1$\frac{1}{8}$″
9×36	1,725,000	143,850	6.16	3–1$\frac{7}{16}$″	1.54	2–$\frac{7}{8}$″	4.93	4–1$\frac{1}{8}$″
10×20	564,000	47,000	4.10	4–1″	1.03	2–$\frac{3}{4}$″	3.28	3–1$\frac{1}{16}$″
10×22	692,500	57,750	4.43	4–1$\frac{1}{16}$″	1.11	2–$\frac{3}{4}$″	3.54	3–1$\frac{1}{16}$″

TABLE Ib. — Continued.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span. (In lower side of girder.)		Reinforcement, outside spans. (In lower side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
10×24	830,000	69,200	4.90	4–1⅛ "	1.23	2– $\frac{13}{16}$"	3.92	3–1⅛ "
10×26	975,000	81,250	5.12	4–1⅛ "	1.28	2– $\frac{13}{16}$"	4.10	3–1¼ "
10×28	1,142,000	95,200	5.40	3–1⅜ "	1.35	2– $\frac{13}{16}$"	4.32	4–1$\frac{1}{16}$"
10×30	1,312,000	109,500	5.73	3–1$\frac{7}{16}$"	1.44	2– ⅞ "	4.58	4–1⅛ "
10×32	1,500,000	125,000	6.09	3–1½ "	1.52	2– ⅞ "	4.87	4–1⅛ "
10×34	1,707,000	142,200	6.71	3–1½ "	1.68	2– $\frac{15}{16}$"	5.37	4–1$\frac{3}{16}$"
10×36	1,914,000	159,600	6.80	3–1$\frac{9}{16}$"	1.70	2– $\frac{15}{16}$"	5.45	4–1$\frac{3}{16}$"
10×38	2,127,000	177,400	7.14	3–1$\frac{9}{16}$"	1.79	2–1 "	5.71	4–1¼ "
10×40	2,380,000	198,500	7.35	3–1⅝ "	1.84	2–1 "	5.88	4–1¼ "
11×22	752,000	63,500	4.97	5–1 "	1.24	2– $\frac{13}{16}$"	3.98	4–1 "
11×24	910,000	75,800	5.36	5–1$\frac{1}{16}$"	1.34	2– $\frac{13}{16}$"	4.29	4–1$\frac{1}{16}$"
11×26	1,074,000	89,200	5.68	5–1$\frac{1}{16}$"	1.42	2– ⅞ "	4.55	4–1$\frac{1}{16}$"
11×28	1,253,000	104,500	6.02	4–1¼ "	1.51	2– ⅞ "	4.82	4–1⅛ "
11×30	1,447,000	120,500	6.48	4–1¼ "	1.62	2– $\frac{15}{16}$"	5.18	5–1$\frac{1}{16}$"
11×32	1,655,000	138,000	6.65	3–1½ "	1.66	2– $\frac{15}{16}$"	5.32	5–1$\frac{1}{16}$"
11×34	1,875,000	156,000	7.07	3–1$\frac{9}{16}$"	1.77	2– $\frac{15}{16}$"	5.65	5–1⅛ "
11×36	2,106,000	175,500	7.45	3–1⅝ "	1.87	2–1 "	5.96	5–1⅛ "
11×38	2,340,000	194,600	7.80	3–1$\frac{11}{16}$"	1.95	2–1 "	6.25	5–1⅛ "
11×40	2,615,000	217,500	8.24	3–1$\frac{11}{16}$"	2.06	2–1$\frac{1}{16}$"	6.60	4–1$\frac{5}{16}$"
11×42	2,890,000	240,800	8.61	3–1¾ "	2.15	2–1$\frac{1}{16}$"	6.89	4–1$\frac{5}{16}$"
11×44	3,170,000	264,000	8.95	3–1¾ "	2.24	2–1$\frac{1}{16}$"	7.16	4–1⅜ "
12×24	994,000	82,800	5.58	5–1$\frac{1}{16}$"	1.40	2– ⅞ "	4.47	4–1$\frac{1}{16}$"
12×26	1,170,000	97,500	6.08	4–1¼ "	1.52	2– ⅞ "	4.87	4–1⅛ "
12×28	1,370,000	114,000	6.51	4–1$\frac{5}{16}$"	1.63	2– $\frac{15}{16}$"	5.21	5–1$\frac{1}{16}$"
12×30	1,578,000	131,500	6.97	4–1$\frac{5}{16}$"	1.74	2– $\frac{15}{16}$"	5.57	5–1$\frac{1}{16}$"
12×32	1,800,000	150,000	7.33	4–1⅜ "	1.83	2–1 "	5.87	5–1⅛ "
12×34	2,050,000	170,700	7.61	4–1$\frac{7}{16}$"	1.90	2–1 "	6.10	5–1⅛ "
12×36	2,300,000	191,600	8.12	3–1$\frac{11}{16}$"	2.03	2–1 "	6.49	5–1$\frac{3}{16}$"
12×38	2,540,000	211,500	8.42	3–1¾ "	2.10	2–1$\frac{1}{16}$"	6.73	4–1$\frac{5}{16}$"

TABLE I*b*. — Continued.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span. (In lower side of girder.)		Reinforcement, outside spans. (In lower side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
12×40	2,816,000	234,000	8.92	3-1$\frac{3}{4}$"	2.23	2-1$\frac{1}{16}$"	7.13	4-1$\frac{3}{8}$"
12×42	3,080,000	256,400	9.35	3-1$\frac{13}{16}$"	2.34	2-1$\frac{1}{8}$"	7.48	4-1$\frac{3}{8}$"
12×44	3,352,000	279,000	9.85	3-1$\frac{13}{16}$"	2.46	2-1$\frac{1}{8}$"	7.88	4-1$\frac{7}{16}$"
12×46	3,620,000	301,700	10.25	3-1$\frac{7}{8}$"	2.56	3-$\frac{15}{16}$"	8.20	4-1$\frac{7}{16}$"
12×48	3,970,000	332,000	10.75	3-1$\frac{7}{8}$"	2.70	3-1"	8.60	4-1$\frac{1}{2}$"
13×26	1,270,000	105,800	6.60	5-1$\frac{3}{16}$"	1.65	2-$\frac{15}{16}$"	5.28	5-1$\frac{1}{16}$"
13×28	1,484,000	124,000	7.05	5-1$\frac{3}{16}$"	1.76	2-1"	5.65	5-1$\frac{1}{16}$"
13×30	1,710,000	142,500	7.68	5-1$\frac{1}{4}$"	1.92	2-1"	6.15	5-1$\frac{1}{8}$"
13×32	1,950,000	162,500	8.08	5-1$\frac{5}{16}$"	2.02	2-1"	6.47	5-1$\frac{1}{4}$"
13×34	2,214,000	184,400	8.49	5-1$\frac{5}{16}$"	2.12	2-1$\frac{1}{16}$"	6.80	5-1$\frac{1}{4}$"
13×36	2,493,000	207,700	8.92	4-1$\frac{1}{2}$"	2.23	2-1$\frac{1}{16}$"	7.14	4-1$\frac{3}{8}$"
13×38	2,767,000	230,200	9.19	4-1$\frac{1}{2}$"	2.40	2-1$\frac{1}{8}$"	7.36	4-1$\frac{3}{8}$"
13×40	3,095,000	258,000	9.72	4-1$\frac{9}{16}$"	2.43	2-1$\frac{1}{8}$"	7.78	4-1$\frac{7}{16}$"
13×42	3,413,000	284,300	10.16	4-1$\frac{5}{8}$"	2.54	3-$\frac{15}{16}$"	8.14	4-1$\frac{7}{16}$"
13×44	3,765,000	313,800	10.46	3-1$\frac{7}{8}$"	2.61	3-$\frac{15}{16}$"	8.38	3-1$\frac{11}{16}$"
13×46	3,940,000	328,000	11.06	3-1$\frac{15}{16}$"	2.77	3-1"	8.85	3-1$\frac{3}{4}$"
13×48	4,300,000	358,600	11.47	3-1$\frac{15}{16}$"	2.87	3-1"	9.17	3-1$\frac{3}{4}$"
13×50	4,710,000	392,800	11.98	3-2"	2.99	3-1"	9.58	3-1$\frac{13}{16}$"
13×52	5,085,000	422,500	12.39	3-2$\frac{1}{16}$"	3.10	3-1$\frac{1}{16}$"	9.92	3-1$\frac{7}{8}$"
14×28	1,593,000	133,000	7.62	5-1$\frac{1}{4}$"	1.91	2-1"	6.15	5-1$\frac{1}{8}$"
14×30	1,840,000	152,700	8.20	5-1$\frac{5}{16}$"	2.05	2-1$\frac{1}{16}$"	6.56	5-1$\frac{3}{16}$"
14×32	2,100,000	174,000	8.68	5-1$\frac{3}{8}$"	2.17	2-1$\frac{1}{16}$"	6.95	5-1$\frac{3}{16}$"
14×34	2,385,000	198,000	8.97	4-1$\frac{1}{2}$"	2.24	2-1$\frac{1}{16}$"	7.18	4-1$\frac{3}{8}$"
14×36	2,680,000	222,400	9.48	4-1$\frac{9}{16}$"	2.37	2-1$\frac{1}{8}$"	7.60	4-1$\frac{3}{8}$"
14×38	2,970,000	246,000	9.89	4-1$\frac{5}{8}$"	2.47	2-1$\frac{1}{8}$"	7.92	4-1$\frac{7}{16}$"
14×40	3,282,000	272,400	10.47	4-1$\frac{5}{8}$"	2.62	3-$\frac{15}{16}$"	8.38	4-1$\frac{7}{16}$"
14×42	3,602,000	299,000	10.93	4-1$\frac{11}{16}$"	2.73	3-1"	8.75	4-1$\frac{1}{2}$"
14×44	3,920,000	324,600	11.42	4-1$\frac{11}{16}$"	2.86	3-1"	9.14	4-1$\frac{9}{16}$"
14×46	4,238,000	349,000	11.77	3-2"	2.94	3-1"	9.43	3-1$\frac{13}{16}$"

TABLE I*b*. — Continued.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span. (In lower side of girder.)		Reinforcement, outside spans. (In lower side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
14 × 48	4,625,000	381,200	12.36	3–2 1/16″	3.09	3–1 1/16″	9.89	3–1 13/16″
14 × 50	5,055,000	417,000	12.85	3–2 1/16″	3.21	3–1 1/16″	10.29	3–1 7/8 ″
14 × 52	5,465,000	450,100	13.27	3–2 1/8 ″	3.32	3–1 1/16″	10.61	3–1 7/8 ″
14 × 54	5,918,000	487,500	13.72	3–2 1/8 ″	3.43	3–1 1/8 ″	10.99	3–1 15/16″
14 × 56	6,385,000	526,500	14.20	3–2 3/16″	3.55	3–1 1/8 ″	11.37	3–2 ″
15 × 30	1,975,000	165,000	8.57	4–1 1/2 ″	2.14	3– 7/8 ″	6.86	4–1 5/16″
15 × 32	2,230,000	186,000	8.97	4–1 1/2 ″	2.24	3– 7/8 ″	7.18	4–1 3/8 ″
15 × 34	2,560,000	213,500	9.62	4–1 9/16″	2.41	3– 15/16″	7.70	4–1 3/8 ″
15 × 36	2,872,000	239,600	10.10	4–1 5/8 ″	2.53	3– 15/16″	8.08	4–1 7/16″
15 × 38	3,186,000	266,000	10.54	4–1 5/8 ″	2.63	3– 15/16″	8.40	4–1 1/2 ″
15 × 40	3,520,000	294,000	11.19	4–1 11/16″	2.80	3–1 ″	8.95	4–1 1/2 ″
15 × 42	3,860,000	322,000	11.66	4–1 11/16″	2.92	3–1 ″	9.32	4–1 9/16″
15 × 44	4,195,000	350,000	12.18	4–1 3/4 ″	3.05	3–1 ″	9.74	4–1 9/16″
15 × 46	4,527,000	378,000	12.75	4–1 13/16″	3.19	3–1 1/16″	10.20	4–1 5/8 ″
15 × 48	4,960,000	414,000	13.27	4–1 13/16″	3.32	3–1 1/16″	10.60	4–1 11/16″
15 × 50	5,420,000	452,000	13.84	3–2 1/8 ″	3.46	3–1 1/8 ″	11.06	3–1 15/16″
15 × 52	5,845,000	485,000	14.23	3–2 3/16″	3.56	3–1 1/8 ″	11.37	3–2 ″
15 × 54	6,350,000	529,500	14.88	3–2 1/4 ″	3.72	3–1 1/8 ″	11.89	3–2 ″
15 × 56	6,849,000	570,500	15.37	3–2 1/4 ″	3.84	4–1 ″	12.29	3–2 1/16″
15 × 58	7,370,000	615,000	15.80	3–2 5/16″	3.95	4–1 ″	12.63	3–2 1/16″
15 × 60	7,872,000	656,000	16.23	3–2 3/8 ″	4.06	4–1 ″	12.97	3–2 1/8 ″
16 × 32	2,402,000	200,000	9.75	5–1 7/16″	2.44	3– 15/16″	7.80	5–1 5/16″
16 × 34	2,724,000	226,800	9.84	5–1 7/16″	2.46	3– 15/16″	7.87	5–1 5/16″
16 × 36	3,070,000	256,000	10.91	5–1 1/2 ″	2.73	3–1 ″	8.73	5–1 3/8 ″
16 × 28	3,402,000	283,300	11.35	5–1 1/2 ″	2.84	3–1 ″	9.08	5–1 3/8 ″
16 × 40	3,762,000	313,500	11.86	4–1 3/4 ″	2.97	3–1 ″	9.48	4–1 9/16″
16 × 42	4,125,000	343,400	12.38	4–1 3/4 ″	3.10	3–1 1/16″	9.90	4–1 5/8 ″
16 × 44	4,480,000	373,500	12.97	4–1 13/16″	3.24	3–1 1/16″	10.10	4–1 5/8 ″
16 × 46	4,850,000	403,000	13.62	4–1 7/8 ″	3.41	3–1 1/16″	10.60	4–1 5/8 ″

TABLE Ib. — Continued.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span. (In lower side of girder.)		Reinforcement, outside spans (In lower. side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
16×48	5,290,000	440,200	14.14	4–$1\frac{7}{8}$″	3.54	3–$1\frac{1}{8}$″	11.00	4–$1\frac{11}{16}$″
16×50	5,790,000	482,000	14.65	3–$2\frac{1}{4}$″	3.66	3–$1\frac{1}{8}$″	11.70	3–2″
16×52	6,250,000	520,500	15.13	3–$2\frac{1}{4}$″	3.78	4–1″	12.10	3–2″
16×54	6,770,000	564,000	15.65	3–$2\frac{5}{16}$″	3.91	4–1″	12.50	3–$2\frac{1}{16}$″
16×56	7,310,000	609,500	16.25	3–$2\frac{3}{8}$″	4.06	4–1″	13.00	3–$2\frac{1}{16}$″
16×58	7,850,000	654,900	16.76	3–$2\frac{3}{8}$″	4.19	4–$1\frac{1}{16}$″	13.40	3–$2\frac{1}{8}$″
16×60	8,400,000	700,000	17.34	3–$2\frac{7}{16}$″	4.34	4–$1\frac{1}{16}$″	13.86	3–$2\frac{1}{8}$″
16×62	9,000,000	750,000	17.80	3–$2\frac{7}{16}$″	4.45	4–$1\frac{1}{16}$″	14.23	3–$2\frac{3}{16}$″
16×64	9,600,000	800,000	18.36	3–$2\frac{1}{2}$″	4.59	4–$1\frac{1}{8}$″	14.67	3–$2\frac{3}{16}$″
17×34	2,886,000	241,000	10.90	5–$1\frac{1}{2}$″	2.73	3–1″	8.72	5–$1\frac{3}{8}$″
17×36	3,258,000	271,500	11.40	5–$1\frac{1}{2}$″	2.85	3–1″	9.12	5–$1\frac{3}{8}$″
17×38	3,620,000	301,400	12.04	5–$1\frac{9}{16}$″	3.01	3–1″	9.62	5–$1\frac{7}{16}$″
17×40	4,040,000	336,500	12.70	5–$1\frac{5}{8}$″	3.18	3–$1\frac{1}{16}$″	10.15	5–$1\frac{7}{16}$″
17×42	4,470,000	372,500	13.22	5–$1\frac{5}{8}$″	3.31	3–$1\frac{1}{16}$″	10.56	5–$1\frac{1}{2}$″
17×44	4,900,000	408,600	13.86	5–$1\frac{11}{16}$″	3.47	3–$1\frac{1}{16}$″	11.09	5–$1\frac{1}{2}$″
17×46	5,130,000	427,500	14.40	4–$1\frac{15}{16}$″	3.60	3–$1\frac{1}{8}$″	11.51	4–$1\frac{3}{4}$″
17×48	5,620,000	468,500	15.00	4–$1\frac{15}{16}$″	3.75	3–$1\frac{1}{8}$″	12.00	4–$1\frac{3}{4}$″
17×50	6,140,000	512,200	15.60	4–2″	3.90	4–1″	12.47	4–$1\frac{13}{16}$″
17×52	6,635,000	554,000	16.11	4–2″	4.03	4–1″	12.88	4–$1\frac{13}{16}$″
17×54	7,200,000	600,000	16.85	4–$2\frac{1}{16}$″	4.21	4–$1\frac{1}{16}$″	13.48	4–$1\frac{7}{8}$″
17×56	7,750,000	646,000	17.20	3–$2\frac{7}{16}$″	4.30	4–$1\frac{1}{16}$″	13.75	3–$2\frac{1}{8}$″
17×58	8,350,000	698,500	17.74	3–$2\frac{7}{16}$″	4.44	4–$1\frac{1}{16}$″	14.20	3–$2\frac{3}{16}$″
17×60	8,925,000	744,500	18.30	3–$2\frac{1}{2}$″	4.58	4–$1\frac{1}{8}$″	14.63	3–$2\frac{3}{16}$″
17×62	9,550,000	796,000	18.88	3–$2\frac{1}{2}$″	4.72	4–$1\frac{1}{8}$″	15.10	3–$2\frac{1}{4}$″
17×64	10,200,000	850,000	19.50	3–$2\frac{9}{16}$″	4.88	4–$1\frac{1}{8}$″	15.60	3–$2\frac{5}{16}$″
17×66	10,900,000	907,500	20.10	3–$2\frac{9}{16}$″	5.03	4–$1\frac{3}{16}$″	16.08	3–$2\frac{3}{8}$″
17×68	11,560,000	962,500	20.67	3–$2\frac{5}{8}$″	5.17	4–$1\frac{3}{16}$″	16.53	3–$2\frac{3}{8}$″
18×36	3,452,000	288,000	12.10	5–$1\frac{1}{2}$″	3.03	3–1″	9.68	5–$1\frac{7}{16}$″
18×38	3,828,000	319,200	12.64	5–$1\frac{5}{8}$″	3.16	3–$1\frac{1}{16}$″	10.11	5–$1\frac{7}{16}$″

TABLE I*b*. — Continued.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3 5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span. (In lower side of girder.)		Reinforcement, outside spans. (In lower side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
18×40	4,275,000	356,700	13.42	5–1$\frac{11}{16}$″	3.36	3–1$\frac{1}{16}$″	10.74	5–1$\frac{1}{2}$ ″
18×42	4,725,000	394,000	14.04	5–1$\frac{11}{16}$″	3.51	3–1$\frac{1}{8}$ ″	11.23	5–1$\frac{1}{2}$ ″
18×44	5,205,000	434,500	14.53	4–1$\frac{15}{16}$″	3.63	3–1$\frac{1}{8}$ ″	11.62	4–1$\frac{3}{4}$ ″
18×46	5,445,000	453,500	15.18	4–1$\frac{15}{16}$″	3.80	3–1$\frac{1}{8}$ ″	12.16	4.1$\frac{3}{4}$ ″
18×48	5,945,000	495,500	15.80	4–2 ″	3.95	4–1 ″	12.65	4–1$\frac{13}{16}$″
18×50	6,500,000	542,200	16.48	4–2$\frac{1}{16}$″	4.12	4–1$\frac{1}{16}$″	13.20	4–1$\frac{13}{16}$″
18×52	7,028,000	586,000	17.12	4–2$\frac{1}{8}$ ″	4.28	4–1$\frac{1}{16}$″	13.70	4–1$\frac{7}{8}$ ″
18×54	7,620,000	636,000	17.75	4–2$\frac{1}{8}$ ″	4.44	4–1$\frac{1}{16}$″	14.20	4–1$\frac{15}{16}$″
18×56	8,210,000	685,000	18.36	4–2$\frac{1}{8}$ ″	4.59	4–1$\frac{1}{8}$ ″	14.69	4–1$\frac{15}{16}$″
18×58	8,830,000	737,000	19.00	4–2$\frac{3}{16}$″	4.75	4–1$\frac{1}{8}$ ″	15.20	4–2 ″
18×60	9,450,000	788,000	19.36	3–2$\frac{9}{16}$″	4.84	4–1$\frac{1}{8}$ ″	15.50	3–2$\frac{5}{16}$″
18×62	10,130,000	843,500	19.94	3–2$\frac{5}{8}$ ″	4.99	4–1$\frac{1}{8}$ ″	15.95	3–2$\frac{5}{16}$″
18×64	10,800,000	900,000	20.52	3–2$\frac{5}{8}$ ″	5.13	4–1$\frac{3}{16}$″	16.40	3–2$\frac{3}{8}$ ″
18×66	11,550,000	961,000	21.25	3–2$\frac{11}{16}$″	5.31	4–1$\frac{3}{16}$″	17.00	3–2$\frac{7}{16}$″
18×68	12,230,000	1,020,000	21.84	3–2$\frac{11}{16}$″	5.46	4–1$\frac{3}{16}$″	17.46	3–2$\frac{7}{16}$″
18×70	13,040,000	1,089,000	22.50	3–2$\frac{3}{4}$ ″	5.63	4–1$\frac{1}{4}$ ″	18.00	3–2$\frac{1}{2}$ ″
18×72	13,800,000	1,150,000	23.05	3–2$\frac{13}{16}$″	5.76	4–1$\frac{1}{4}$ ″	18.43	3–2$\frac{1}{2}$ ″
19×38	4,030,000	336,200	13.37	5–1$\frac{11}{16}$″	3.34	3–1$\frac{1}{16}$″	10.70	5–1$\frac{1}{2}$ ″
19×40	4,520,000	376,900	14.10	5–1$\frac{11}{16}$″	3.53	3–1$\frac{1}{8}$ ″	11.28	5–1$\frac{9}{16}$″
19×42	4,980,000	415,000	14.78	5–1$\frac{3}{4}$ ″	3.70	3–1$\frac{1}{8}$ ″	11.82	5–1$\frac{9}{16}$″
19×44	5,495,000	458,800	15.43	5–1$\frac{3}{4}$ ″	3.86	4–1 ″	12.33	5–1$\frac{5}{8}$ ″
19×46	5,745,000	478,700	16.20	5–1$\frac{13}{16}$″	4.05	4–1 ″	12.94	5–1$\frac{5}{8}$ ″
19×48	6,291,000	524,800	16.77	4–2$\frac{1}{16}$″	4.19	4–1$\frac{1}{16}$″	13.40	4–1$\frac{7}{8}$ ″
19×50	6,855,000	571,000	17.42	4–2$\frac{1}{8}$ ″	4.36	4–1$\frac{1}{16}$″	13.94	4–1$\frac{7}{8}$ ″
19×52	7,430,000	618,400	18.00	4–2$\frac{1}{8}$ ″	4.50	4–1$\frac{1}{16}$″	14.40	4–1$\frac{15}{16}$″
19×54	8,038,000	670,000	18.73	4–2$\frac{3}{16}$″	4.68	4–1 ″	14.97	4–1$\frac{15}{16}$″
19×56	8,655,000	722,000	19.32	4–2$\frac{1}{4}$ ″	4.83	4–1$\frac{1}{8}$ ″	15.45	4–2 ″
19×58	9,325,000	777,700	20.05	4–2$\frac{1}{4}$ ″	5.01	4–1$\frac{1}{8}$ ″	16.03	4–2 ″
19×60	9,978,000	832,500	20.58	4–2$\frac{5}{16}$″	5.15	5–1$\frac{1}{16}$″	16.46	4–2$\frac{1}{16}$″
19×62	10,700,000	892,000	21.10	3–2$\frac{11}{16}$″	5.23	5–1$\frac{1}{16}$″	16.88	3–2$\frac{7}{16}$″

TABLE I*b*. — Continued.

1	2	3	4	5	6	7	8	9
Size of beam.	Safe bending moment. Factor of safety = 3.5.		Reinforcement over intermediate supports. (In upper side of girder.)		Reinforcement, intermediate span. (In lower side of girder.)		Reinforcement, outside spans. (In lower side of girder.)	
			Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.	Area of metal.	No. and size of bars.
In.	In. lbs.	Ft. lbs.	Sq. in.		Sq. in.		Sq. in.	
19×64	11,420,000	950,000	21.75	3-2$\frac{11}{16}$″	5.44	5-1$\frac{1}{16}$″	17.40	3-2$\frac{7}{16}$″
19×66	12,180,000	1,015,000	22.40	3-2$\frac{3}{4}$″	5.60	5-1$\frac{1}{16}$″	17.90	3-2$\frac{1}{2}$″
19×68	12,940,000	1,088,000	23.00	3-2$\frac{13}{16}$″	5.75	5-1$\frac{1}{8}$″	18.40	3-2$\frac{1}{2}$″
19×70	13,750,000	1,145,000	23.68	3-2$\frac{13}{16}$″	5.92	5-1$\frac{1}{8}$″	18.94	3-2$\frac{9}{16}$″
19×72	14,565,000	1,213,000	24.35	3-2$\frac{7}{8}$″	6.09	5-1$\frac{1}{8}$″	19.46	3-2$\frac{9}{16}$″
19×74	15,420,000	1,286,000	25.00	3-2$\frac{15}{16}$″	6.25	5-1$\frac{1}{8}$″	20.00	3-2$\frac{5}{8}$″
19×76	16,245,000	1,353,500	25.68	3-2$\frac{15}{16}$″	6.42	5-1$\frac{3}{16}$″	20.53	3-2$\frac{5}{8}$″
20×40	4,770,000	396,800	14.80	5-1$\frac{3}{4}$″	3.70	4-1″	11.85	5-1$\frac{9}{16}$″
20×42	5,250,000	437,500	15.46	5-1$\frac{3}{4}$″	3.87	4-1″	12.37	5-1$\frac{5}{8}$″
20×44	5,795,000	482,000	16.20	5-1$\frac{13}{16}$″	4.05	4-1″	12.97	5-1$\frac{5}{8}$″
20×46	6,050,000	504,700	16.94	5-1$\frac{7}{8}$″	4.24	4-1$\frac{1}{16}$″	13.56	5-1$\frac{11}{16}$″
20×48	6,625,000	552,000	17.72	5-1$\frac{15}{16}$″	4.43	4-1$\frac{1}{16}$″	14.19	5-1$\frac{11}{16}$″
20×50	7,235,000	602,800	18.96	5-1$\frac{15}{16}$″	4.61	4-1$\frac{1}{8}$″	14.77	5-1$\frac{3}{4}$″
20×52	7,820,000	651,200	19.02	5-1$\frac{15}{16}$″	4.76	4-1$\frac{1}{8}$″	15.22	4-2″
20×54	8,600,000	717,000	19.71	4-2$\frac{1}{4}$″	4.93	4-1$\frac{1}{8}$″	15.79	4-2″
20×56	9,126,000	761,000	20.26	4-2$\frac{1}{4}$″	5.07	5-1″	16.24	4-2$\frac{1}{16}$″
20×58	9,828,000	819,200	21.00	4-2$\frac{5}{16}$″	5.25	5-1$\frac{1}{16}$″	16.81	4-2$\frac{1}{16}$″
20×60	10,500,000	875,000	21.60	4-2$\frac{3}{8}$″	5 40	5-1$\frac{1}{16}$″	17.30	4-2$\frac{1}{8}$″
20×62	11,250,000	937,700	22.32	4-2$\frac{3}{8}$″	5.58	5-1$\frac{1}{16}$″	17.88	4-2$\frac{1}{8}$″
20×64	12,040,000	1,003,000	23.03	4-2$\frac{7}{16}$″	5.76	5-1$\frac{1}{8}$″	18.44	4-2$\frac{3}{16}$″
20×66	12,830,000	1,067,000	23.71	4-2$\frac{7}{16}$″	5.93	5-1$\frac{1}{8}$″	19.00	3-2$\frac{9}{16}$″
20×68	13,600,000	1,145,000	24.15	3-2$\frac{7}{8}$″	6.04	5-1$\frac{1}{8}$″	19.34	3-2$\frac{9}{16}$″
20×70	14,510,000	1,208,000	24.95	3-2$\frac{15}{16}$″	6.24	5-1$\frac{1}{8}$″	19.98	3-2$\frac{5}{8}$″
20×72	15,340,000	1,278,000	25.60	3-2$\frac{15}{16}$″	6.40	5-1$\frac{3}{16}$″	20.50	3-2$\frac{5}{8}$″
20×74	16,250,000	1,353,000	26.22	3-3″	6.56	5-1$\frac{3}{16}$″	21.00	3-2$\frac{11}{16}$″
20×76	17,140,000	1,428,000	26.86	3-3″	6.72	5-1$\frac{3}{16}$″	21 50	3-2$\frac{11}{16}$″
20×78	18,100,000	1,508,000	27.66	3-3$\frac{1}{16}$″	6.92	5-1$\frac{3}{16}$″	22.14	3-2$\frac{3}{4}$″
20×80	19,090,000	1,588,000	28.37	3-3$\frac{1}{8}$″	7.09	5-1$\frac{1}{4}$″	22.71	3-2$\frac{3}{4}$″

Description of Table II.

This table is inserted to be used in a more specific way than Table I, in cases of uniform loading where the total live load in tons uniformly distributed along the beam or girder is known, as well as the span in feet. Only a sufficient number of sizes are here given to cover the ordinary loading met with in practice, and such sizes are selected as are capable of withstanding the severest loading for a given amount of material — in other words, the most economical sizes. It may happen in practice that certain local conditions, such as want of head-room, etc., may enter the case to an extent to prohibit the use of certain sizes here given. For such cases Table I, which is more general in scope, may be resorted to. Then again, in certain particular cases, neither of the two tables may apply. In such instances, which can happen only seldom, the designer may well afford to spend his time to meet the special requirements.

The table in itself needs little, if any, explanation. Column 1 gives the span in feet; column 2 gives the gross load in tons uniformly distributed along the span, given in column 1, that the size, designated above, will safely carry with a factor of safety of 3.5. Column 3, in a like manner, gives the net load after deducting the weight of the beam itself.

Only one other thing needs mentioning. It may be noticed that opposite two different spans of

each size, the corresponding loading is underscored. This is to show that all spans lying between the underscoring are the proper ones to use whenever possible, for a cantilever loading only, in order to be certain that the shearing stress brought to bear upon the section will not be excessive. (Table III takes into account the design to resist various shearing values.) It should be noted that the loading for all spans above the higher limit, designated by the underscoring, can safely be used with a cantilever loading, but less economically, because, for the sizes of shear bars designated in Table III, there will be supplied more metal than is necessary to withstand the shear, allowing the same factor of safety of 3.5. In other words, above this limit, we are designing safely, but not as economically, as possible. At the same time, other sizes for the particular span may be selected, which will be safe as well as economical. On the other hand, it is not safe to use the loading for spans below the limit designated by the underscoring, without increasing the area of the shear bars over the largest size given in Table III. In all cases, for loadings with supported or fixed ends, the tables apply safely.

TABLE II. — *Beams and Girders (Single Spans Supported at Ends).*

	2½″ × 6″		2½″ × 8″		2½″ × 10″		2½″ × 12″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
3	.87	.85	......	......	......	......	......	......
4	.65	.62	1.28	1.24	2.10	2.05	......	......
5	.52	.48	1.02	.97	1.68	1.62	2.52	2.44
6	.43	.38	.85	.79	1.40	1.32	2.10	2.01
7	.37	.32	.73	.66	1.20	1.11	1.80	1.69
8	.32	.26	.64	.56	1.05	.95	1.58	1.46
9	......	......	.51	.49	.93	.82	1.40	1.27
10	......	......	.51	.42	.84	.71	1.26	1.11
11	......	......	......	......	.76	.62	1.14	.97
12	......	......	......	......	.70	.55	1.05	.87
13	......	......	......	......	.65	.49	.97	.77
14	......	......	......	......	......	......	.90	.69
15	......	......	......	......	......	......	.84	.62
	3″ × 12″		3″ × 14″		4″ × 14″		4″ × 16″	
5	3.03	2.94	......	......	......	......	......	......
6	2.53	2.42	3.53	3.40	4.69	4.52	......	......
7	2.16	2.03	3.08	2.93	4.02	3.82	5.36	5.13
8	1.90	1.75	2.65	2.47	3.52	3.29	4.69	4.43
9	1.69	1.52	2.35	2.15	3.13	2.87	4.17	3.87
10	1.52	1.33	2.12	1.90	2.82	2.53	3.75	3.42
11	1.38	1.17	1.92	1.68	2.56	2.24	3.41	3.05
12	1.26	1.03	1.81	1.55	2.35	2.00	3.13	2.73

TABLE II. — *Beams and Girders.* — Continued.

	3″ × 12″		3″ × 14″		4″ × 14		4″ × 16″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
13	1.17	.93	1.63	1.34	2.16	1.78	2.89	2.46
14	1.08	.82	1.51	1.20	2.01	1.60	2.68	2.22
15	1.01	.73	1.41	1.08	1.88	1.44	2.50	2.00
16	....	.....	1.32	.97	1.76	1.29	2.35	1.82
17	......	......	1.24	.87	1.66	1.17	2.21	1.65
18	......	......	1.18	.79	1.56	1.04	2.09	1.49
19	......	......	......	......	......	....	1.98	1.35
20	......	......	......	......	......	....	1.88	1.22
	5″ × 16″		5″ × 18″		5″ × 20″		6″ × 20″	
7	6.69	6.40	....	......	....	....	....	......
8	5.86	5.53	7.50	7.12	9.40	8.98	11.27	10.77
9	5.20	4.82	6.47	6.05	8.34	7.87	10.04	9.48
10	4.68	4.26	6.00	5.53	7.52	7.00	9.02	8.39
11	4.26	3.80	5.46	4.94	6.84	6.26	8.21	7.52
12	3.95	3.45	5.01	4.45	6.26	5.63	7.52	6.77
13	3.61	3.07	4.62	4.01	5.78	5.10	6.95	6.13
14	3.35	2.77	4.30	3.65	5.37	4.64	6.45	5.51
15	3.13	2.50	4.00	3.30	5.01	4.23	6.02	5.08
16	2.93	2.26	3.76	3.01	4.70	3.86	5.64	4.64
17	2.76	2.05	3.53	2.73	4.42	3.58	5.31	4.25
18	2.61	1.86	3.34	2.50	4.18	3.24	5.02	3.89
19	2.47	1.68	3.16	2.27	3.96	2.97	4.75	3.56
20	2.34	1.51	3.00	2.06	3.76	2.71	4.52	3.27
21	....	......	2.86	1.88	3.58	2.48	4.30	2.98
22	....	......	2.78	1.71	3.41	2.26	4.10	2.72
23	....	......	2.61	1.48	3.27	2.07	3.93	2.49
24	....	......	....	......	3.13	1.87	3.76	2.26
25	....	......	....	......	3.00	1.69	3.61	2.04

TABLE II. — *Beams and Girders.* — Continued.

	6″ × 22″		6″ × 24″		7″ × 24″		7″ × 26″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
9	12.30	11.78	....	.....	....	....	....	......
10	11.08	10.39	13.37	12.62	15.48	14.62	18.25	17.30
11	10.07	9.31	12.16	11.33	14.08	13.12	16.60	15.56
12	9.22	8.39	11.15	10.25	12.90	11.85	15.20	14.06
13	8.52	7.62	10.30	9.32	11.90	10.76	14.04	12.81
14	7.90	6.93	9.55	8.50	11.05	9.83	13.04	11.71
15	7.38	6.34	8.91	7.78	10.32	9.31	12.17	10.75
16	6.92	5.81	8.37	7.17	9.67	8.27	11.41	9.89
17	6.52	5.35	7.87	6.59	9.10	7.61	10.73	9.12
18	6.15	4.91	7.43	6.08	8.60	7.02	10.14	8.43
19	5.83	4.52	7.04	5.61	8.13	6.47	9.62	7.75
20	5.54	4.16	6.68	5.18	7.73	5.98	9.12	7.22
21	5.27	3.82	6.38	4.80	7.38	5.54	8.70	6.71
22	5.03	3.51	6.08	4.43	7.03	5.10	8.30	6.21
23	4.82	3.23	5.82	4.09	6.73	4.72	7.94	5.76
24	4.61	2.95	5.58	3.78	6.45	4.35	7.61	5.33
25	4.43	2.70	5.35	3.47	6.19	4.00	7.30	4.92
26	4.26	2.46	5.14	3.19	5.97	3.50	7.02	4.55
27	4.10	2.23	4.95	2.92	5.73	3.37	6.77	4.21
28	....	......	4.78	2.68	5.53	3.08	6.52	3.86
29	....	......	4.61	2.43	5.34	2.80	6.30	3.54
30	....	......	4.46	2.20	5.16	2.53	6.08	3.13
31	....	......	....	....	....	....	5.90	2.96
32	....	......	....	....	....	....	5.70	2.66

TABLE II. — *Beams and Girders.* — Continued.

	7″ × 28″		8″ × 28″		8″ × 30″		8″ × 32″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
11	19.40	18.28	21.95	20.66	......	......	......	......
12	17.76	16.53	20.10	18.70	23.35	21.85	......	......
13	16.40	15.07	18.57	17.05	21.55	19.93	24.62	22.88
14	15.20	13.77	17.26	16.63	20.00	18.25	22.85	20.98
15	14.20	12.67	16.10	14.45	18.67	16.80	21.30	19.30
16	13.32	11.69	15.10	13.23	17.50	15.50	20.00	17.86
17	12.54	10.81	14.20	12.22	16.46	14.34	18.82	16.55
18	11.84	10.01	13.42	11.32	15.56	13.31	17.78	15.38
19	11.22	9.28	12.71	10.49	14.75	12.37	16.86	14.32
20	10.65	8.61	12.08	9.74	14.00	11.50	16.00	13.33
21	10.16	8.02	11.50	9.05	13.33	10.71	15.25	12.45
22	9.68	7.44	10.97	8.40	12.74	9.99	14.56	11.62
23	9.27	6.93	10.50	7.81	12.17	9.30	13.92	10.85
24	8.88	6.44	10.06	7.26	11.68	8.68	13.34	10.14
25	8.52	5.97	9.66	6.74	11.20	8.08	12.80	9.46
26	8.20	5.55	9.28	6.24	10.77	7.52	12.30	8.83
27	7.90	5.15	8.95	5.80	10.37	7.00	11.87	8.27
28	7.62	4.77	8.63	5.36	10.00	6.50	11.44	7.70
29	7.35	4.39	8.33	4.94	9.65	6.03	11.04	7.17
30	7.10	4.04	8.06	4.56	9.34	5.59	10.68	6.68
31	6.88	3.72	7.78	4.16	9.04	5.16	10.32	6.19
32	6.66	3.40	7.55	3.81	8.75	4.75	10.00	5.73
33	6.46	3.10	7.32	3.36	8.48	4.36	9.70	5.30
34	6.27	2.80	7.10	3.13	8.22	3.97	9.41	4.87
35	6.10	2.53	6.90	2.81	7.99	3.62	9.15	4.48
36	......	......	......	......	7.78	3.28	8.90	4.10
37	......	......	......	......	7.54	2.92	8.67	3.74
38	......	......	......	......	......	......	8.43	3.36
39	......	......	......	......	......	......	8.21	3.01
40	......	......	......	......	......	......	8.00	2.67

TABLE II. — *Beams and Girders.* — Continued.

	9″ × 32″		9″ × 34″		9″ × 36″			
1	2	3	2	3	2	3	2	
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
13	27.80	25.85	....	......	....	....	....	......
14	25.80	23.70	29.30	26.97	....	....	....	......
15	24.08	21.83	27.27	24.88	30.68	28.15	....	......
16	22.55	20.15	25.55	23.00	28.72	26.02	....	......
17	21.25	18.70	24.04	21.33	27.08	24.21	....	......
18	20.05	17.30	22.70	19.83	25.58	22.54	....	......
19	19.00	16.15	21.52	18.49	24.22	21.01	....	......
20	18.08	15.08	20.43	17.24	23.02	19.64	....	......
21	17.20	14.05	19.50	16.15	21.94	18.40	....	......
22	16.40	13.10	18.59	15.08	20.92	17.21	....	......
23	15.70	12.25	17.78	14.11	20.00	16.12	....	......
24	15.08	11.48	17.03	13.20	19.17	15.12	....	......
25	14.45	10.70	16.36	12.37	18.40	14.18	....	......
26	13.88	9.98	15.74	11.59	17.71	13.32	....	......
27	13.37	9.32	15.15	10.85	17.04	12.49	....	......
28	12.90	8.70	14.52	10.06	16.45	11.73	....	......
29	12.45	8.10	14.10	9.48	15.87	10.98	....	......
30	12.03	7.53	13.63	8.86	15.35	10.29	....	......
31	11.54	6.89	13.20	8.26	14.84	9.61	....	......
32	11.29	6.49	12.78	7.68	14.40	9.00	....	......
33	10.94	5.99	12.39	7.13	13.94	8.37	....	......
34	10.52	5.42	12.04	6.62	13.54	7.80	....	......
35	10.31	5.06	11.69	6.11	13.15	7.25	....	......
36	10.04	4.64	11.36	5.62	12.80	6.73	....	......
37	9.76	4.21	11.06	5.16	12.44	6.20	....	......
38	9.51	3.71	10.76	4.71	12.12	5.71	....	......
39	9.25	3.40	10.49	4.27	11.80	5.22	....	......
40	9.02	3.02	10.23	3.86	11.50	4.85	....	......
41	....	......	9.98	3.45	11.22	4.30	....	......
42	....	......	9.74	3.04	10.96	3.88	....	......
43	....	......	....	......	10.70	3.45	....	......
44	....	......	....	......	10.47	3.05	....	......
45	....	......	....	......	10.24	2.64	....	......

TABLE II. — *Beams and Girders.* — Continued.

	10″ × 36″		10″ × 38″		10″ × 40″		11″ × 40″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
16	31.90	28.90	35.45	32.28	......	......	......	......
18	28.35	24.97	31.52	27.66	35.30	31.56	38.75	34.63
20	25.50	21.75	28.34	24.28	31.75	27.59	34.83	30.25
22	23.20	19.07	25.80	21.45	28.88	24.30	31.70	26.66
24	21.27	17.77	23.65	18.90	26.48	21.49	29.05	23.55
26	19.64	14.77	21.84	16.69	24.46	18.55	26.82	20.67
28	18.24	12.99	20.28	14.73	22.70	16.87	24.92	17.52
30	17.00	11.36	18.90	12.95	21.18	14.94	23.24	16.37
32	15.94	9.94	17.74	11.41	19.86	13.21	21.80	14.48
34	15.00	8.64	16.70	9.97	18.70	11.63	20.50	12.72
36	14.18	7.41	15.77	8.65	17.56	10.07	19.39	11.15
38	13.44	6.31	14.94	7.42	16.72	8.82	18.35	9.65
40	12.76	5.26	14.20	6.28	15.90	7.58	17.40	8.25
42	12.16	4.28	13.50	5.19	15.13	6.40	16.60	7.00
44	11.60	3.35	12.90	4.20	14.44	5.29	15.85	5.79
46	......	......	12.35	3.25	13.81	4.26	15.16	4.54
48	......	......	11.83	2.33	13.24	3.27	14.54	3.56
50	......	......	......	......	12.70	2.30	13.95	2.51

	11″ × 42″		11″ × 44″		12″ × 44″		12″ × 46″	
18	42.80	38.46	47.05	42.46	49.70	44.85	......	......
20	38.50	33.68	42.35	37.30	44.75	39.25	48.30	42.55
22	35.02	29.72	38.50	32.95	40.70	34.65	43.86	37.53
24	32.10	26.32	35.27	29.22	37.30	30.70	40.25	33.35
26	29.64	23.38	32.58	26.03	34.47	27.32	37.10	29.62
28	27.54	20.79	30.22	23.17	32.00	24.30	34.52	26.47
30	25.70	18.47	28.22	20.65	29.82	21.57	32.20	23.58
32	24.10	16.40	26.44	18.37	28.00	19.20	30.25	21.05

TABLE II. — *Beams and Girders.* — Continued.

	11″ × 42″		11″ × 44″		12″ × 44″		12″ × 46″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
34	22.67	14.47	24.90	16.33	26.33	16.98	28.40	18.63
36	21.41	12.83	23.50	14.43	24.88	14.98	26.80	16.45
38	20.28	11.13	22.28	12.71	23.53	12.94	25.40	14.46
40	19.27	9.64	21.15	11.05	22.36	11.36	24.15	13.65
42	18.35	8.21	20.16	9.56	21.32	9.77	23.00	10.92
44	17.50	6.90	19.24	8.14	20.34	8.24	21.93	9.28
46	16.75	5.65	18.40	6.80	19.46	6.82	20.98	7.75
48	16.06	4.47	17.63	5.53	18.65	5.45	20.10	6.20
50	15.43	3.27	16.93	4.30	17.87	4.12	19.30	4.90
52	14.83	2.27	16.27	3.15	17.21	2.91	18.58	3.63
54	....	......	15.67	3.03	16.60	1.75	17.89	2.36
56	....	......	....	......	....	....	17.26	1.16
58	....	......	....	......	....	....	16.76	.06
	12″ × 48″		**13″ × 48″**		**13″ × 50″**		**13″ × 52″**	
20	53.00	47.00	55.30	48.80	....	....	....	......
22	48.20	41.50	50.28	43.13	57.25	49.80	61.50	53.76
24	44.20	37.00	46.20	38.40	52.45	44.30	56.40	47.96
26	40.77	32.97	42.55	34.10	48.45	39.60	52.05	42.91
28	37.87	29.47	39.54	30.44	45.00	35.52	48.30	38.46
30	35.30	26.30	36.90	27.15	42.00	31.85	45.15	34.60
32	33.12	23.50	34.60	24.20	39.33	28.49	42.30	31.05
34	31.17	20.97	32.57	21.52	37.00	25.48	39.80	27.85
36	29.42	18.62	30.78	19.08	34.96	22.76	37.60	24.95
38	27.90	16.50	29.13	16.78	33.12	20.26	35.60	22.25
40	26.50	14.50	27.70	14.70	31.48	17.93	33.87	19.80
42	25.20	12.60	26.35	12.70	29.95	15.71	32.22	17.45
44	24.08	10.88	25.16	11.86	28.60	13.50	30.80	15.33
46	23.05	9.25	24.07	9.12	27.36	11.78	29.44	13.28

TABLE II. — *Beams and Girders.* — Continued.

	12″ × 48″		13″ × 48″		13″ × 50″		13″ × 52″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
48	22.10	7.70	23.06	7.46	26.20	9.95	28.20	11.33
50	21.20	6.20	22.15	5.90	25.18	8.24	27.10	9.54
52	20.38	4.78	21.30	4.40	24.20	6.60	26.06	7.77
54	19.63	3.43	20.53	2.98	23.30	5.00	25.10	6.10
56	18.94	2.14	19.76	1.56	22.47	3.51	24.20	4.54
58	18.28	.88	19.08	.33	21.72	2.08	23.34	2.94
60	17.67	......	18.47	......	21.00	.70	22.58	1.48
62	......	......	......	......	20.30	......	21.85	......
	14″ × 52″		14″ × 54″		14″ × 56″		15″ × 56″	
22	66.30	57.96	......	......	......	......	......	......
24	60.80	51.70	65.90	56.47	71.20	61.40	76.00	65.47
26	56.20	46.36	60.80	50.57	65.70	55.08	70.20	58.80
28	52.20	41.60	56.50	45.50	61.00	49.55	65.20	52.92
30	48.70	37.33	52.75	40.90	57.00	44.74	60.80	47.65
32	45.65	33.53	49.75	37.18	53.35	40.27	57.03	43.00
34	42.95	30.08	46.50	33.14	50.23	36.30	53.72	38.82
36	40.60	26.98	44.00	29.85	47.50	32.80	50.75	34.95
38	38.43	24.03	41.65	26.72	44.98	29.45	48.00	31.34
40	36.50	21.36	39.52	23.82	42.70	26.35	45.65	28.61
42	34.75	19.85	37.65	21.15	40.70	23.54	43.50	25.10
44	33.20	16.53	35.95	18.67	38.80	20.80	41.50	22.20
46	31.73	14.33	34.40	16.35	37.13	18.30	39.70	19.52
48	30.40	12.22	32.95	14.10	35.60	16.00	38.00	17.95
50	29.22	10.29	31.65	12.01	34.18	13.76	36.50	14.58
52	28.08	8.41	30.40	10.00	32.82	11.56	35.10	12.22
54	27.06	6.64	29.28	8.08	31.60	9.52	33.80	10.12
56	26.08	4.88	28.26	6.26	30.50	7.60	32.60	8.05
58	25.18	3.23	27.28	4.48	29.42	5.70	31.50	6.08
60	24.32	1.62	26.37	2.72	28.45	3.90	30.43	4.13
62	23.53	.07	25.50	1.15	27.55	2.25	29.45	2.25
64	......	......	24.70	......	26.70	.55	28.53	.45

TABLE II. — *Beams and Girders.* — Continued.

	15″ × 58″		15″ × 60″		16″ × 60″		16″ × 62″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
24	82.00	71.13	....	......	....	....	....	......
26	75.80	64.03	80.80	68.62	86.20	73.20	92.40	78.98
28	70.30	57.63	75.00	61.89	80.00	66.00	85.80	71.34
30	65.60	52.00	70.00	53.94	74.75	59.75	80.10	64.60
32	61.50	47.00	65.70	50.70	70.05	54.05	75.10	58.58
34	57.90	42.50	61.77	45.83	66.00	49.00	70.70	53.15
36	54.70	38.40	58.40	41.54	62.30	44.30	66.75	48.15
38	51.80	34.60	55.25	37.45	59.00	40.00	63.20	43.57
40	49.25	31.15	52.50	33.76	56.00	36.00	60.00	39.35
42	46.80	27.80	50.00	30.32	53.30	32.30	57.20	35.50
44	44.75	24.85	47.75	27.13	51.00	29.00	54.65	31.95
46	42.78	21.95	45.70	24.15	48.75	25.75	52.30	28.55
48	41.00	19.38	43.75	21.27	46.70	22.70	50.00	25.20
50	39.40	16.78	42.00	18.58	44.90	19.90	48.00	22.20
52	37.90	14.35	40.40	16.02	43.15	17.15	46.25	19.41
54	36.44	11.99	38.90	13.60	41.50	14.50	44.50	16.60
56	35.20	10.82	37.50	11.25	40.00	12.00	42.90	14.00
58	33.95	7.70	36.20	9.00	38.70	9.70	41.45	11.50
60	32.80	5.60	35.00	6.90	37.40	7.40	40.00	9.00
62	31.74	3.66	33.90	4.87	36.20	5.20	38.80	6.80
64	30.78	1.78	32.86	2.86	35.08	3.08	37.55	4.50
66	29.85	......	31.80	.85	34.00	1.00	36.40	2.30
68	....	......	30.92	......	33.00	....	35.40	.40

	16″ × 64″		17″ × 64″		17″ × 66″		17″ × 68″	
28	91.70	76.78	97.20	81.36	103.70	87.40	110.30	93.50
30	85.50	69.50	90.75	73.80	96.80	79.30	103.00	85.00
32	80.25	63.20	85.00	66.90	90.75	72.11	96.50	77.30
34	75.50	57.40	80.00	60.80	85.30	65.50	90.80	70.40
36	71.30	52.10	75.60	55.26	80.70	59.70	85.75	64.15

TABLE II. — *Beams and Girders.* — Continued.

	16″ × 64″		17″ × 64″		17″ × 66″		17″ × 68″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
38	67.60	47.34	71.60	50.12	76.45	54.30	81.30	58.50
40	64.20	42.88	68.00	45.40	72.60	49.30	77.10	53.10
42	61.00	38.60	64.75	41.02	69.15	44.65	73.50	48.30
44	58.30	34.86	61.80	36.92	66.00	40.38	70.20	43.80
46	55.75	31.25	59.15	33.15	63.10	36.30	67.10	39.50
48	53.50	27.90	56.70	29.58	60.50	32.50	64.30	35.50
50	51.35	24.70	54.35	26.30	58.20	29.10	61.75	31.75
52	49.30	21.60	52.30	22.90	55.80	25.50	59.35	28.15
54	47.55	18.80	50.40	19.90	53.80	22.34	57.20	24.80
56	45.80	15.97	48.50	16.85	51.80	19.20	55.15	21.55
58	44.25	13.33	46.85	14.10	50.05	16.25	53.25	17.45
60	42.75	10.75	45.35	11.45	48.35	12.35	51.50	15.50
62	41.45	8.41	43.85	8.85	46.80	9.68	49.80	12.60
64	40.15	6.05	42.50	6.30	45.40	8.10	48.25	9.85
66	38.90	3.70	41.20	3.90	44.00	5.57	46.75	7.15
68	37.75	1.51	40.00	1.60	42.70	3.10	45.40	4.60
70	36.70	......	38.90	......	41.50	.75	44.15	3.15
72	....	......	....	......	40.30	....	42.90	......
	18″ × 68″		18″ × 70″		18″ × 72″		19″ × 72″	
28	116.50	98.65	....	......	....	....	....	......
30	108.90	89.76	116.50	96.80	122.80	102.53	129.50	118.05
32	102.00	81.60	109.00	88.00	115.00	93.40	121.50	98.62
34	96.00	74.30	102.50	80.20	108.40	85.45	114.30	90.00
36	90.75	67.75	96.90	73.28	102.50	78.20	108.00	82.28
38	85.90	61.68	91.75	66.82	96.90	71.25	102.40	75.22
40	81.60	56.10	87.10	52.85	92.00	65.00	97.10	68.50
42	77.70	50.90	83.00	55.40	87.60	59.25	92.50	62.50
44	74.20	46.15	79.20	50.30	83.75	54.05	88.30	56.35
46	71.00	41.84	75.80	45.60	80.20	49.15	84.50	51.64
48	68.00	37.40	72.60	41.10	76.80	44.40	81.00	46.70

TABLE II. — *Beams and Girders.* — Continued.

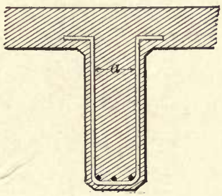
	18″ × 68″		18″ × 70″		18″ × 72″		19″ × 72″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
50	65.20	33.30	69.70	36.90	73.70	39.94	77.75	42.00
52	62.75	29.57	67.00	32.90	70.80	35.70	74.80	37.60
54	60.50	26.08	64.70	29.25	68.20	31.72	72.00	33.40
56	58.25	24.55	62.25	25.47	65 80	28.00	69.40	29.64
58	56.30	19.30	60.00	21.92	63.50	24.30	67.00	25.50
60	54.50	16.25	58.10	18.60	61.30	20.80	64.75	21.85
62	52.65	13.15	56.30	15.60	59.35	17.50	62.70	18.40
64	51.00	10.22	54.50	12.50	57.50	14.30	60.75	15.00
66	49.40	8.30	52.80	9.50	55.90	11.40	58.90	11.70
68	48.00	4.65	51.25	6.55	54.20	8.30	57.20	8.55
70	46.55	1.88	49.80	3.80	52.70	5.45	55.50	5.50
72	45.30	......	48.50	1.25	51.20	2.60	54.00	2.52
74	....	.....	....	......	49.75	....	52.50	......
	19″ × 74″		19″ × 76″		20″ × 76″		20″ × 78″	
30	137.00	114.94	....	......	....	....	....	......
32	128.60	105.10	135.20	111.02	142.70	117.30	150.70	124.70
34	121.00	96.00	127.40	101.72	134.20	107.25	141.80	114.20
36	114.40	87.95	120.40	93.20	127.70	99.15	134.00	104.80
38	108.40	80.50	114.00	85.30	120.00	89.88	126.80	95.95
40	103.00	73.60	108.20	78.00	114.20	82.50	120.50	88.00
42	98.00	67.12	103.00	71.30	108.70	75.40	115.00	80.90
44	93.50	61.15	98.50	65.30	103.70	68.82	109.50	73.75
46	89.50	55.70	94.00	59.45	99.20	62.75	104.90	67.50
48	85.80	50.50	90.25	54.00	95.10	56.05	100.50	61.50
50	82.25	45.50	86.70	48.95	91.25	51.65	96.40	55.80
52	79.20	41.00	83.25	43.98	87.80	46.55	92.80	50.55
54	76.25	36.55	80.20	39.45	84.50	41.75	89.25	45.35
56	73.50	32.30	77.30	35.05	81.50	37.15	86.10	40.60
58	71.00	28.40	74.70	30.95	78.70	32.70	83.10	36.00
60	68.50	24.40	72.20	26.92	76.00	28.50	80.30	31.55
62	66.30	20.75	69.80	22.92	73.70	24.55	77.80	26.45

TABLE II. — *Beams and Girders.* — Continued.

	19″ × 74″		19″ × 76″		20″ × 76″		20″ × 78″	
1	2	3	2	3	2	3	2	3
Span.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.	Load Gross.	Load Net.
64	64.25	17.25	67.75	19.45	71.30	20.55	75.25	23.25
66	62.30	13.80	65.70	15.90	69.20	16.92	73.00	19.40
68	60.60	10.60	63.75	12.40	67.20	13.30	70.90	15.65
70	58.75	7.25	61.80	8.90	65.20	9.70	68.80	11.90
72	57.20	4.25	60.20	5.80	63.30	6.30	67.00	8.50
74	55.70	1.30	58.50	2.60	61.70	3.00	65.20	5.05
76	54.20	......	57.00	......	60.00	....	63.40	1.70
78	....	......	....	......	....	....	61.80	......
	20″ × 80″							
34	149.20	120.90	....	......	....	....	....	......
36	141.00	111.00	....	......	....	....	....	......
38	133.50	101.90	....	......	....	....	....	......
40	127.00	93.70	....	......	....	....	....	......
42	120.80	85.80	....	......	....	....	....	......
44	115.60	79.00	....	......	....	....	....	......
46	110.40	72.10	....	......	....	....	....	......
48	105.60	65.60	....	......	....	....	....	......
50	101.50	59.90	....	......	....	....	....	......
52	97.50	54.20	....	......	....	....	....	......
54	94.00	49.00	....	......	....	....	....	......
56	90.50	43.90	....	......	....	....	....	......
58	87.50	39.20	....	......	....	....	....	......
60	84.60	34.60	....	......	....	....	....	......
62	81.80	30.20	....	......	....	....	....	......
64	79.20	25.90	....	......	....	....	....	......
66	76.80	21.80	....	......	....	....	....	......
68	74.60	18.00	....	......	....	....	....	......
70	72.50	14.20	....	......	....	....	....	......
72	70.50	10.50	....	......	....	....	....	......
74	68.50	6.90	....	......	....	....	....	......
76	66.75	3.45	....	......	....	....	....	......
78	65.00	......	....	......	....	....	....	......

Description of Table III.

Table III is given to show how much shearing force the same size of beams, figured to resist bending in Table I, will resist. Suppose a certain size has been selected from Tables I or II to withstand the maximum bending moment in the case at hand. Of course we know the maximum shearing force in this particular case, for this had to be determined before obtaining the maximum bending moment. Hence, by referring to Table III, and opposite the size already selected, ascertain under which of the three columns, 5, 6, or 7, the

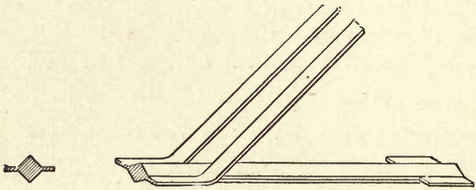


SKETCH 1.

shearing force in question falls. As you will note by the titles of these columns, column 5 is figured to use shear bars with an aggregate area of cross-section of .19 square inches, column 6 with an area of .28 square inches; likewise column 7 of .38 square inches area.

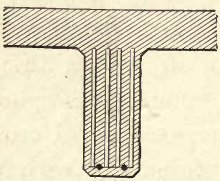
These shear bars are found upon the market of various designs. Very commonly they are in the form of a U-shaped bar lettered "a" in Sketch 1, which may be inserted vertically to the horizontal axis of the beams, or inclined at an angle, prop-

erly 60 degrees, with the vertical axis, and in the direction at right angles to the lines of shear cracks, which develop when a beam is

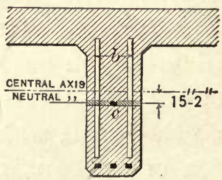


SKETCH 2.

tested to destruction, as may be seen in cuts of two tests shown in another section on page 45. These may be met with in the rolled and stamped section shown in Sketch 2 and inserted in the beam as shown in Sketch 3. These are two of several patented shapes, which are used to resist longitudinal shear. Instead of making the tables apply to any of these special shapes, the writer



SKETCH 3



SKETCH 4.

has taken a general case, shown in Sketch 4, to which results any of the patented shapes may be applied.

By the aggregate area of shear bars just mentioned, is meant the combined area of the cross-sections of bars lettered "b" in Sketch 4. Bars

"b" should be placed at an angle of 60 degrees, about with the vertical.

In the tables, column 1 gives the size of beams; column 2 gives the total area of the size opposite which it appears; column 3 gives the area of concrete, and 4 the steel area, making up the total area under column 2.

VERTICAL SHEAR. — In obtaining the values under column 5, it was reasoned that the maximum shearing force, which always happens at a support, causes a given strain upon the section at that point, which strain is uniform at all points of the section, through the steel, as well as through the concrete. The value of this strain was so fixed that the stress per square inch caused thereby, throughout the concrete section, was very moderate, which always happens with a concentrated load in the middle of the span of a beam supported at both ends. The strain just referred to was fixed at .0000167 inches, which, with a modulus of elasticity of 3,000,000, gives a working stress of a 1–1½–3 or a 1–2–4 concrete, 50 pounds per square inch, and a factor of safety of 7, calling the ultimate shearing stress 350 pounds per square inch.

Column 6 was figured allowing the working strain and the stress caused thereby to be moderate, and is adapted to meet the ordinary cases of a beam uniformly loaded and supported at the ends. The working stress was fixed at .000025 inches, which in a like manner means a working

stress of 75 pounds per square inch, or a factor of safety of 5.25.

In a like manner column 7 was figured to be adapted to general cases of cantilever loading, where the maximum shearing force, for a given maximum bending moment, is greatest. In this case the strain was limited to .0000333 inches, giving a corresponding working stress of 100 pounds per square inch, and a factor of safety of 3.5.

Hence it may be seen that column 5 is generally applicable to cases of concentrated central loads upon beams supported at both ends; column 6 to cases of uniform loading upon beams similarly supported; and column 7 to cases of cantilever loading. This adaptation holds only in a general way, and will not apply to all cases, one of which was mentioned under Table II.

To obtain any shearing force under column 5, the area of the concrete in square inches given under column 3 was multiplied by 50 pounds per square inch, to which force in pounds, was added the product of multiplying the area of steel in square inches under column 4 by 500 pounds per square inch, since by applying a given strain to both concrete and steel, there is caused ten times the stress in the steel as there is in the concrete. In a like manner the values under columns 6 and 7 were obtained by using 75 and 100 pounds per square inch allowable stress for the concrete, and the proportionate stresses of 750 and 1,000 pounds per square inch for the area of steel respectively.

Longitudinal Shear. — It is a well recognized fact that, in elastic beams undergoing vertical shearing, there is caused a corresponding longitudinal shear which is greatest at the neutral axis of the section in question, and decreases at any axis approaching either the top or bottom of the section. In rectangular shapes, the intensity of stress at the neutral axis equals 3 $\times$ the total vertical shearing force divided by $2 \times$ the breadth of the beam $\times$ the depth of the beam, or, in the characteristic form,

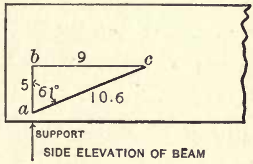
$$\text{Intensity of stress} = \frac{3\,F}{2\,bh}.$$

By applying this to the values of shearing force given under columns 5, 6, or 7, for the breadth and depth of beam given under column 1, we obtain for an intensity of longitudinal shearing stress per square inch of 90 pounds for column 5, 135 pounds for column 6, and 180 pounds for column 7. By comparing these values with the corresponding ones used as working vertical shearing stress per square inch, namely, 50 pounds, 75 pounds, and 100 pounds, you will notice that a given vertical shearing stress causes a longitudinal shearing stress of a magnitude of $\frac{9}{5}$ of itself. Hence, without inserting a steel member to reduce the stress and help out the concrete at the neutral axis of the sections near the supports, we are reducing our working factors of safety by 45 per cent in each case. In other words, instead of

having factors of 7, 5.25, and 3.5, as we had for the three particular cases in vertical shear, we have corresponding ones of but 3.9, 2.92, and 1.95, which are not ample, and if they were, would be unsatisfactory to use, since the general design would be weaker in some places than others. To relieve the concrete at this point, we have only to insert a rod of an area of $\frac{1}{18}$ of that of the concrete in the layer where the width of the layer is the side of the rod, and the length is the width of the beam. See rod *C*, Sketch 4, which is the rod just mentioned, and the shaded section, which represents the concrete. The rod should be $\frac{1}{18}$, because we have to relieve the stress in the concrete by 45 per cent, and the effect of a given area of steel is ten times that of a like area of concrete. By combining these two ratios, we get $\frac{1}{10} \times \frac{5}{9} = \frac{1}{18}$ This means for a 5-inch wide beam, that rod *C* should be $\frac{1}{4}$ inch square; for a 10-inch wide beam, $\frac{1}{2}$ inch square; and for a 20-inch wide beam, 1 inch square. All intervening sizes may be graded accordingly. As this rod *C* is not required at the middle of the span, and is most needed at the ends, it may not be continuous, but used at the ends only, and extend sufficiently toward the center to satisfy the designer that the longitudinal shearing stress beyond the limits of the rod is not excessive.

It has already been stated that a given vertical shearing stress causes a longitudinal shearing stress of $\frac{9}{5}$ of itself. Hence in Sketch 5, if we lay

off to any scale, *ab* in a vertical line and equal to five parts, and *bc* to the same scale equal to nine parts, and at right angles to *ab*, then will the line *ac* give the magnitude of the resultant when divided into the same units that the other two forces were laid out to, and also the direction of action of this resultant. The magnitude, as will be noted, becomes 2.1 times the vertical shearing stress, and the direction of action is inclined to the vertical at an angle *bac*, the tangent of which is 1.8, which denotes an angle of 61 degrees. It has been remarked that the shear-cracks, when a beam is tested to destruction, occur along lines making an angle of about 60 degrees with the vertical. The above demonstration goes to show why such is the case.



SKETCH 5.

It might have been argued, upon first thought, that putting in bar *c*, Sketch 3, could have no effect upon the longitudinal shear and would in no wise help out the concrete tending to shear in a plane parallel with itself, but upon referring to Sketch 4, we see that rod *c* will cut the resultant line of force *ac* at an angle of 29 degrees and, consequently can offer a component to react against the resultant *ac*.

The rods marked "b" in Sketch 4 will be sufficient to help out the concrete at all axes except the neutral axis, for the aggregate areas of these

rods will, when .19 square inch area, and the width of beam 20 inches, which is the limit of the table, reduce the longitudinal shearing stress by 10 per cent, which stress reduces in magnitude greatly as the top or bottom layers are approached. Ten per cent increase over the factor of safety of 3.9 gives 4.3 at a layer just above or below the neutral axis, which is ample. Again, with an aggregate area of .38 square inches for rods "b," and with a width of beam of 20 inches, the shearing stress would be reduced 19 per cent, and the factor of safety at a layer just either side of the neutral axis would be 2.3 at the very least, and undoubtedly much more, which may be ascertained by applying the formulæ for longitudinal shearing stress at that layer.

In distributing the shear bars along the length of a beam or girder the following suggestion is offered. Since the shearing stress per square inch varies uniformly from zero at the free end of a cantilever, or at the center between the supports or fixed ends of a beam, to a maximum at the supports or fixed ends, the spacing of bars should vary inversely and uniformly. Because this variation is uniform, it may be represented by the relationship of the odd numbers 1, 3, 5, 7, etc., to each other. With a uniformly distributed loading along the length of the beam or girder, the shearing stress per square inch varies as the length of a cantilever beam called "l," and as the half length also called "l," of a beam supported or

fixed at the ends. Likewise the shearing stress varies indirectly as the depth of the beam designated "*d*." Hence the shearing stress varies as the ratio of $\frac{l}{d}$. This value increases when approaching the supports and since, as stated before, the spacing of bars decreases, this latter may be represented by the inverse ratio or $\frac{d}{l}$. Accordingly it is suggested to call this ratio a fraction of a foot, then express its value in inches, thus giving the location of the first pair of bars from the supports or fixed ends. The location of the next pair, in a like manner, is three times this constant from the first pair; likewise the third pair are distant five times this constant from the second pair and so on, receding from the support.

Note.

In dealing with continuous girders, the values of the safe shearing forces given in columns 5, 6, and 7 of Table III, should be modified as follows:

With 2 spans decrease the values given by 12.5 per cent.

With 3 spans decrease the values given by 11.0 per cent.

With 4 spans decrease the values given by 11.5 per cent.

With 5, 6, 7, and 8 spans, decrease the values given by 11.3 per cent.

TABLE III.

1	2	3	4	5	6	7
Size of beam.	Area of beam.	Area of concrete.	Area of steel.	Shearing force in tons.		
				Area of shear bars 0.19 in.	Area of shear bars 0.28 in.	Area of shear bars 0.38 in.
2.5×6	15	14.53	.47	.48	.72	.96
2.5×8	20	19.48	.52	.62	.93	1.24
2.5×10	25	24.35	.65	.77	1.16	1.54
2.5×12	30	29.43	.66	.90	1.35	1.80
3×6	18	17.52	.48	.56	.84	1.12
3×8	24	23.41	.59	.73	1.10	1.47
3×10	30	29.34	.66	.90	1.35	1.80
3×12	36	35.22	.78	1.08	1.59	2.15
3×14	42	41.12	.88	1.25	1.87	2.50
4×8	32	31.25	.75	.97	1.45	1.94
4×10	40	39.14	.86	1.19	1.79	2.39
4×12	48	47.01	.99	1.43	2.14	2.85
4×14	56	54.86	1.14	1.66	2.49	3.31
4×16	64	62.74	1.26	1.86	2.79	3.77
5×10	50	48.85	1.15	1.51	2.26	3.02
5×12	60	58.68	1.32	1.80	2.70	3.60
5×14	70	68.54	1.46	2.08	3.12	4.16
5×16	80	78.32	1.68	2.38	3.32	4.76
5×18	90	88.14	1.86	2.67	4.00	5.34
5×20	100	97.94	2.06	2.97	4.50	5.93
6×12	72	70.47	1.53	2.14	3.22	4.29
6×14	84	82.22	1.78	2.50	3.75	5.00
6×16	96	94.03	1.97	2.84	4.26	5.69
6×18	108	105.81	2.19	3.20	4.79	6.39
6×20	120	117.60	2.40	3.54	5.31	7.08
6×22	132	129.38	2.62	3.89	5.81	7.78
6×24	144	141.16	2.84	4.24	6.36	8.48
7×14	98	95.86	2.14	2.94	4.41	5.87
7×16	112	109.59	2.41	3.34	5.02	6.69
7×18	126	123.38	2.62	3.73	5.60	7.48

TABLE III. — Continued.

1	2	3	4	5	6	7
				Shearing force in tons.		
Size of beam.	Area of beam.	Area of concrete.	Area of steel.	Area of shear bars 0.19 in.	Area of shear bars 0.28 in.	Area of shear bars 0.38 in.
7×20	140	137.13	2.87	4.14	6.22	8.29
7×22	154	150.89	3.11	4.55	6.83	9.10
7×24	168	164.73	3.27	4.93	7.40	9.87
7×26	182	178.47	3.53	5.34	8.02	10.69
7×28	196	192.23	3.77	5.74	8.63	11.50
8×16	128	125.37	2.63	3.79	5.68	7.57
8×18	144	141.10	2.90	4.25	6.37	8.51
8×20	160	156.76	3.24	4.93	7.09	9.46
8×22	176	172.45	3.55	5.19	7.78	10.40
8×24	192	188.18	3.82	5.67	8.50	11.32
8×26	208	203.91	4.09	6.11	9.18	12.24
8×28	224	219.68	4.32	6.56	9.85	13.15
8×30	240	235.35	4.65	7.04	10.57	14.09
8×32	256	251.03	4.97	7.52	11.26	15.04
9×18	162	158.62	3.38	4.81	7.20	9.62
9×20	180	176.27	3.73	5.34	8.02	10.68
9×22	198	193.97	4.03	5.85	8.79	11.71
9×24	216	211.63	4.37	6.38	9.57	12.77
9×26	234	229.35	4.65	6.94	10.84	13.79
9×28	252	247.27	4.73	7.36	11.05	14.73
9×30	270	264.78	5.22	7.93	11.89	15.85
9×32	288	282.46	5.54	8.44	12.68	16.89
9×34	306	300.13	5.87	8.97	13.45	17.94
9×36	324	317.84	6.16	9.48	14.21	18.97
10×20	200	195.90	4.10	5.92	8.89	11.85
10×22	220	215.57	4.43	6.49	9.73	12.99
10×24	240	235.10	4.90	7.10	10.66	14.21
10×26	260	254.88	5.12	7.66	11.47	15.31
10×28	280	274.60	5.40	8.23	12.33	16.43
10×30	300	294.27	5.73	8.78	13.20	17.58
10×32	320	313.91	6.09	9.38	14.04	18.74
10×34	340	333.29	6.71	10.00	15.01	20.02

TABLE III. — Continued.

1	2	3	4	5	6	7
				Shearing force in tons.		
Size of beam.	Area of beam.	Area of concrete.	Area of steel.	Area of shear bars 0.19 in.	Area of shear bars 0.28 in.	Area of shear bars 0.38 in.
10×36	360	353.20	6.80	10.53	15.80	21.06
10×38	380	372.86	7.14	11.10	16.63	22.21
10×40	400	392.47	7.53	11.68	17.53	23.39
11×22	242	237.03	4.97	7.17	10.76	14.34
11×24	264	258.64	5.36	7.79	11.71	15.61
11×26	286	280.32	5.68	8.43	12.64	16.86
11×28	308	301.98	6.02	9.05	13.56	18.11
11×30	330	323.52	6.48	9.70	14.53	19.42
11×32	352	345.35	6.65	10.29	15.45	20.59
11×34	374	366.93	7.07	10.94	16.40	21.88
11×36	396	388.55	7.45	11.58	17.37	23.15
11×38	418	410.20	7.80	12.20	18.33	24.40
11×40	440	431.76	8.24	12.86	19.29	25.71
11×42	462	453.39	8.61	13.48	20.23	26.98
11×44	484	475.05	8.95	14.09	20.91	28.23
12×24	288	282.42	5.58	8.46	12.69	16.91
12×26	312	305.92	6.08	9.17	13.79	18.34
12×28	336	329.49	6.51	9.86	14.79	19.73
12×30	360	353.03	6.97	10.57	15.86	21.14
12×32	384	376.67	7.33	11.26	16.88	22.50
12×34	408	400.39	7.61	11.90	17.86	23.83
12×36	432	423.88	8.12	12.63	18.94	25.26
12×38	456	447.58	8.42	13.31	19.96	26.59
12×40	480	471.08	8.92	13.98	20.99	28.02
12×42	504	494.65	9.35	14.69	22.03	29.41
12×44	528	518.15	9.85	15.41	23.14	30.84
12×46	552	541.75	10.25	16.12	24.14	32.22
12×48	576	565.25	10.75	16.84	25.23	33.64
13×26	338	331.40	6.60	9.95	14.90	19.87
13×28	364	356.95	7.05	10.67	16.00	21.38
13×30	390	382.32	7.68	11.47	17.23	22.96
13×32	416	407.92	8.08	12.22	18.33	24.44

TABLE III. — Continued.

1	2	3	4	5	6	7
Size of beam.	Area of beam.	Area of concrete.	Area of steel.	Shearing force in tons.		
				Area of shear bars 0.19 in.	Area of shear bars 0.28 in.	Area of shear bars 0.38 in.
13×34	442	433.51	8.49	12.95	19.43	25.92
13×36	468	459.08	8.92	13.70	20.55	27.42
13×38	494	484.81	9.19	14.40	21.60	28.84
13×40	520	510.28	9.72	15.20	22.80	30.38
13×42	546	535.84	10.16	15.94	23.91	31.87
13×44	572	561.54	10.46	16.65	24.97	33.31
13×46	598	586.94	11.06	17.94	26.15	34.88
13×48	624	612.53	11.47	18.17	27.30	36.36
13×50	650	638.02	11.98	18.95	28.39	37.89
13×52	676	663.61	12.39	19.69	29.27	39.38
14×28	392	384.38	7.62	11.50	17.27	23.03
14×30	420	411.80	8.20	12.35	18.48	24.69
14×32	448	439.32	8.68	13.15	19.72	26.32
14×34	476	467.03	8.97	13.91	20.86	27.84
14×36	504	494.52	9.48	14.72	22.08	29.47
14×38	532	522.11	9.89	15.50	23.25	31.05
14×40	560	549.53	10.47	16.37	24.54	32.71
14×42	588	577.07	10.93	17.14	25.73	34.32
14×44	616	604.58	11.42	17.98	26.91	35.94
14×46	644	632.23	11.77	18.74	28.11	37.50
14×48	672	659.64	12.36	19.56	29.37	39.16
14×50	700	687.15	12.85	20.37	30.57	40.78
14×52	728	714.73	13.27	21.17	31.73	42.37
14×54	756	742.28	13.72	21.95	32.92	43.98
14×56	785	770.80	14.20	22.85	34.23	45.64
15×30	450	441.43	8.57	13.18	19.76	26.36
15×32	480	471.03	8.97	13.99	21.02	28.04
15×34	510	500.38	9.62	14.91	22.36	29.83
15×36	540	529.90	10.10	15.78	23.64	31.55
15×38	570	559.46	10.54	16.61	24.93	33.25
15×40	600	588.81	11.19	17.50	26.29	35.04
15×42	630	618.34	11.66	18.36	27.55	36.75
15×44	660	647.82	12.18	19.23	28.81	38.48

TABLE III. — Continued.

1	2	3	4	5	6	7
				Shearing force in tons.		
Size of beam.	Area of beam.	Area of concrete.	Area of steel.	Area of shear bars 0.19 in.	Area of shear bars 0.28 in.	Area of shear bars 0.38 in.
15×46	690	677.25	12.75	20.09	30.15	40.24
15×48	720	706.71	13.27	20.97	31.48	41.97
15×50	750	736.16	13.84	21.86	32.79	43.73
15×52	780	765.77	14.23	22.71	34.09	45.41
15×54	810	795.12	14.88	23.61	35.43	47.20
15×56	840	824.63	15.37	24.44	36.67	48.92
15×58	870	854.20	15.80	25.30	37.93	50.61
15×60	900	883.77	16.23	26.15	39.21	52.31
16×32	512	502.25	9.75	14.99	22.50	29.99
16×34	544	534.16	9.84	15.81	23.69	31.63
16×36	576	565.09	10.91	16.87	25.29	33.71
16×38	608	596.65	11.35	17.74	26.63	35.51
16×40	640	628.14	11.86	18.67	27.97	37.34
16×42	672	659.62	12.38	19.57	29.37	39.17
16×44	704	691.03	12.97	20.52	30.64	41.04
16×46	736	722.38	13.62	21.45	32.19	42.93
16×48	768	753.86	14.14	22.34	33.54	44.77
16×50	800	785.35	14.65	23.31	34.95	46.59
16×52	832	816.87	15.13	24.19	36.27	48.41
16×54	864	848.35	15.65	25.11	37.37	50.25
16×56	896	879.75	16.25	26.04	39.06	52.12
16×58	928	911.24	16.76	26.94	40.42	53.94
16×60	960	942.96	17.04	27.83	41.77	55.67
16×62	992	974.20	17.80	28.82	43.18	57.61
16×64	1024	1005.64	18.36	29.72	44.64	59.46
17×34	578	567.10	10.90	16.88	25.35	33.81
17×36	612	600.60	11.40	17.85	26.79	35.73
17×38	646	633.96	12.04	18.86	28.26	37.72
17×40	680	667.30	12.70	19.88	29.76	39.72
17×42	714	700.78	13.22	20.80	31.21	41.65
17×44	748	734.19	13.86	21.82	32.70	43.64
17×46	782	767.60	14.40	22.79	34.15	45.58
17×48	816	801.00	15.00	23.75	35.66	47.55

TABLE III. — Continued.

1	2	3	4	5	6	7
Size of beam.	Area of beam.	Area of concrete.	Area of steel.	Shearing force in tons.		
				Area of shear bars 0.19 in.	Area of shear bars 0.28 in.	Area of shear bars 0.38 in.
17 × 50	850	834.40	15.60	24.75	37.11	49.52
17 × 52	884	867.89	16.11	25.75	38.57	51.45
17 × 54	928	911.15	16.85	26.97	39.03	53.98
17 × 56	962	944.80	17.20	27.90	41.82	55.84
17 × 58	996	978.26	17.74	28.90	43.33	57.79
17 × 60	1030	1011.70	18.30	29.84	44.80	59.74
17 × 62	1064	1045.12	18.88	30.86	46.31	61.70
17 × 64	1098	1078.50	19.50	31.83	49.70	63.68
17 × 66	1132	1111.90	20.10	32.80	49.43	65.65
17 × 68	1166	1145.33	20.67	33.80	50.75	67.60
18 × 36	648	635.90	12.10	18.92	28.39	37.85
18 × 38	684	761.36	12.64	19.96	29.89	39.89
18 × 40	720	706.58	13.42	21.01	31.54	42.04
18 × 42	756	741.96	14.04	22.05	33.07	44.12
18 × 44	792	777.47	14.53	23.04	34.58	46.14
18 × 46	828	812.82	15.18	24.10	35.81	48.23
18 × 48	864	848.20	15.80	25.15	37.68	50.31
18 × 50	900	883.52	16.48	26.22	39.28	52.42
18 × 52	936	918.88	17.12	27.27	40.85	54.51
18 × 54	972	954.25	17.75	28.32	42.41	56.59
18 × 56	1008	989.64	18.36	29.34	44.00	58.66
18 × 58	1044	1025.00	19.00	30.38	45.53	60.75
18 × 60	1080	1060.64	19.36	31.34	47.00	62.71
18 × 62	1116	1096.06	19.94	32.38	48.58	64.78
18 × 64	1152	1131.48	20.52	33.40	50.17	66.55
18 × 66	1188	1166.75	21.25	34.43	51.66	68.98
18 × 68	1224	1202.16	21.84	35.47	53.24	71.03
18 × 70	1260	1237.50	22.50	36.53	54.82	73.13
18 × 72	1296	1262.95	23.05	37.34	55.92	74.67
19 × 38	722	708.63	13.37	21.04	31.59	42.12
19 × 40	760	745.90	14.10	22.18	33.26	44.35
19 × 42	798	783.22	14.78	23.26	34.93	46.55
19 × 44	836	820.57	15.43	24.35	36.54	48.75
19 × 46	874	857.77	16.23	25.45	38.23	51.01
19 × 48	912	895.23	16.77	26.58	39.87	53.15

TABLE III. — Continued.

1	2	3	4	5	6	7
Size of beam.	Area of beam.	Area of concrete.	Area of steel.	Shearing force in tons.		
				Area of shear bars 0.19 in.	Area of shear bars 0.28 in.	Area of shear bars 0.38 in.
19×50	950	932.58	17.42	27.66	41.53	55.34
19×52	988	970.00	18.00	28.75	43.13	57.50
19×54	1026	1007.27	18.73	29.82	44.76	59.73
19×56	1064	1044.68	19.32	30.93	46.38	61.90
19×58	1102	1081.95	20.05	32.07	48.08	64.13
19×60	1140	1119.42	20.58	33.09	49.62	66.26
19×62	1178	1146.90	21.10	34.15	50.88	67.90
19×64	1216	1194.25	21.75	35.32	52.92	70.59
19×66	1254	1231.60	22.40	36.36	54.55	72.28
19×68	1292	1269.00	23.00	37.35	55.95	74.95
19×70	1330	1306.32	23.68	38.55	51.27	77.16
19×72	1368	1333.65	24.35	39.39	59.08	78.86
19×74	1406	1381.00	25.00	40.75	61.23	81.55
19×76	1444	1418.32	25.68	41.90	62.83	83.76
20×40	800	785.20	14.80	23.35	35.03	46.66
20×42	840	824.54	15.46	24.48	36.72	48.96
20×44	880	863.80	16.20	25.39	38.08	51.29
20×46	920	903.06	16.94	26.79	40.22	53.63
20×48	960	942.28	17.72	28.48	41.95	55.98
20×50	1000	981.04	18.96	29.24	43.87	58.53
20×52	1040	1020.98	19.02	30.25	45.38	60.56
20×54	1080	1060.29	19.71	31.43	47.14	62.87
20×56	1120	1099.74	20.26	32.56	48.83	65.12
20×58	1160	1139.00	21.00	33.70	50.50	67.45
20×60	1200	1178.40	21.60	34.83	52.25	69.72
20×62	1240	1217.68	22.32	35.98	53.99	72.05
20×64	1280	1256.97	23.03	37.14	55.64	74 37
20×66	1320	1296.29	23.71	38.33	57.44	76.67
20×68	1360	1335.85	24.15	39 44	59.16	78.87
20×70	1400	1375.05	24.95	40.62	60.91	81.23
20×72	1440	1414.40	25.60	41.77	62.60	83.52
20×74	1480	1453.78	26.22	42.85	64.33	85.80
20×76	1520	1493.14	26.86	44.00	66.08	88.09
20×78	1560	1532.34	27.66	45.26	67.92	90.45
20×80	1600	1571.63	28.37	46.33	69.53	92.77

Description of Table IV.

This table is worked out for the same cases as was Table II, up to, and including beams 13 inches wide, with the purpose of satisfying the designer that no fear need be felt that the design be weak in resisting deflection. Like designing with steel shapes, little or no attention need be exercised in this regard, unless, in cases where it is expressly desired to obtain and retain a floor strictly level. It was thought that it was unnecessary to carry out the table further, as it may readily be seen that allowing a deflection of $\frac{1}{800}$ of the span, the load causing this deflection, which is expressed in tons uniformly distributed, will be twice as large as will the beam carry with a factor of safety of 3.5, as shown in Table II.

In assigning an allowable deflection of $\frac{1}{800}$ of the span, when working up the table, the writer had two things in mind: first, that this is a very moderate amount, and may well be allowed for floors carrying machinery which has to be maintained level, and in line, and for floors, from the underside of the beams of which is hung shafting; second, for the reason that, at about this deflection, hair cracks begin to appear upon the underside of the beam or girder extending up to the tension members through the non-reinforced protection for the steel. These cracks are of the very slightest importance when considered as affecting the strength of the beam or girder, and

the only objections that can be offered against their existence are the unsightly appearance they offer, and that they render the steel protection less durable as a fire resisting medium. This latter, however, is a matter of great importance, and should be kept well in mind during the design.

The table itself needs no other explanation

NOTE.

Make the following modifications in the values given for safe uniformly distributed loads given under Table IV when applying them to continuous girders:

With 2 spans increase the values by 44.6 per cent.

With 3 spans increase the values by 36.0 per cent.

With 4 spans increase the values by 38.3 per cent.

With 5 spans increase the values by 38.0 per cent.

With 6, 7, 8, and 9 spans, increase the values by 38.0 per cent.

TABLE IV. — *Uniformly Distributed Load in Tons, Allowing a Safe Deflection of $\frac{1}{800}$ of Span, for the Following Sizes.*

Span.	2.5 × 6	2.5 × 8	2.5 × 10	2.5 × 12	3 × 12	3 × 14	4 × 14	4 × 16
5	1.28	3.36	6.85	12.20	14.40	......	......	......
6	.89	2.34	4.76	8.48	10.00	16.40	21.18	......
7	.65	1.72	3.50	6.23	7.36	12.05	15.55	23.70
8	.50	1.32	2.68	4.78	5.75	9.23	12.18	18.15
9	.40	1.04	2.12	3.77	4.45	7.04	9.40	14.35
10	......	.84	1.72	3.05	3.61	5.90	7.62	11.63
11	......	......	1.42	2.53	2.98	4.88	6.30	9.60
12	......	......	1.14	2.03	2.50	4.10	5.29	8.07
13	......	......	1.02	1.81	2.14	3.50	4.51	6.88
14	......	......	......	1.56	1.84	3.02	3.89	5.93
15	......	......	......	1.36	1.60	2.63	3.39	5.16
16	......	......	......	......	......	2.31	2.98	4.54
17	......	......	......	......	......	2.04	2.64	4.02
18	......	......	......	......	......	1.82	2.35	3.59
19	......	......	......	......	......	......	......	3.23
20	......	......	......	......	......	......	......	2.91

Span.	5 × 16	5 × 18	5 × 20	6 × 20	6 × 22	6 × 24	7 × 24	7 × 26
7	31.15	......	......	......	......	......	......	......
8	23.85	33.50	48.10	56.35	......	......	......	......
9	18.90	26.50	37.15	44.50	50.60	......	......	......
10	15.30	21.40	30.10	36.05	41.00	62.50	76.00	......
11	12.60	17.70	24.90	29.78	33.88	52.50	62.75	80.20
12	10.60	14.90	20.90	25.10	28.50	44.10	52.75	67.35
13	9.05	12.70	17.80	21.38	24.30	37.63	45.00	57.50
14	7.79	10.90	15.35	18.38	20.88	32.40	38.75	49.50
15	6.55	9.52	13.40	16.05	18.23	28.25	33.75	43.13
16	6.00	8.38	11.75	14.10	16.00	24.83	29.70	37.90
17	5.28	7.40	10.40	12.50	14.20	22.55	26.25	33.55
18	4.72	6.62	9.29	11.15	12.68	19.60	23.40	29.93
19	4.24	5.94	8.34	10.00	11.38	17.63	21.05	26.90
20	3.82	5.35	7.53	9.03	10.25	15.90	19.00	24.25
21	......	4.87	6.83	8.18	9.30	14.40	17.23	22.00
22	......	4.42	6.23	7.45	8.50	13.15	15.70	20.05
23	......	4.03	5.69	6.78	7.75	12.00	14.35	18.35
24	......	......	5.23	6.28	7.13	11.05	13.20	16.85
25	......	......	4.82	5.77	6.55	10.18	12.15	15.50
26	......	......	......	......	6.08	9.40	11.25	14.35
27	......	......	......	......	5.63	8.73	10.45	13.30

TABLE IV. — *Uniformly Distributed Load.* — Continued.

Span.	5 × 16	5 × 18	5 × 20	6 × 20	6 × 22	6 × 24	7 × 24	7 × 26
28	....	....	....	....	....	8.10	9.70	12.35
29	....	....	....	....	....	7.55	9.03	11.50
30	....	....	....	....	....	7.05	8.43	10.75
31	....	....	....	....	....	....	......	10.08
32	....	......	....	....	....	....	......	9.97
33	....	....	....	....	....	....	......	8.90

Span.	7 × 28	8 × 28	8 × 30	8 × 32	9 × 32	9 × 34	9 × 36	
12	84.50	96.96	120.00	....	....	....	......	......
13	72.00	82.50	102.50	124.50	132.75	......	......	......
14	62.00	71.00	88.25	107.00	114.30	155.00	......	......
15	54.00	62.00	77.00	93.50	99.50	135.00	149.50	......
16	47.50	54.50	67.50	82.00	87.50	118.50	131.50	......
17	42.00	48.25	59.80	72.65	77.75	105.00	116.25	......
18	37.50	43.05	53.35	64.75	69.20	91.50	103.75	......
19	33.75	38.60	48.00	58.25	62.10	84.20	93.50	......
20	30.40	34.85	43.25	52.50	56.00	76.00	84.00	......
21	27.50	31.60	39.20	47.60	50.75	68.85	76.30	......
22	25.13	28.83	35.75	43.45	46.25	62.75	69.50	......
23	23.00	26.38	32.75	39.75	42.35	57.50	63.70	......
24	21.13	23.70	30.05	36.50	38.90	52.75	58.50	......
25	19.45	22.25	27.70	33.63	35.85	48.55	53.90	......
26	18.00	20.60	25.60	31.10	33.15	44.95	49.85	......
27	16.70	19.13	23.75	28.85	30.75	41.75	46.20	......
28	15.50	17.80	22.10	26.80	28.60	38.30	42.90	......
29	14.45	16.55	20.55	25.00	26.60	36.15	40.00	......
30	13.50	15.50	19.20	23.35	24.90	33.75	37.38	......
31	12.65	14.50	18.00	21.85	23.30	31.65	35.00	......
32	11.88	13.60	16.85	20.50	21.87	29.75	32.88	......
33	11.15	12.80	15.90	19.30	20.60	27.95	30.90	......
34	10.50	12.05	15.00	18.20	19.38	26.33	29.13	......
35	9.90	11.35	14.13	17.15	18.30	24.83	27.45	......
36	....	....	13.35	16.20	17.30	23.45	26.00	......
37	....	....	12.65	15.35	16.35	22.20	24.55	......
38	....	....	12.00	14.55	15.50	21.05	23.27	......
39	....	....	....	13.80	14.73	20.00	22.10	......
40	....	....	....	13.13	14.00	19.00	21.00	......
41	....	....	....	....	....	......	20.00	......
42	....	....	....	....	....	......	19.07	......
43	....	....	....	....	....	......	18.18	......
44	....	....	....	....	....	......	17.40	......

TABLE IV. — *Uniformly Distributed Load.* — Continued.

Span.	10 × 36	10 × 38	10 × 40	11 × 40	11 × 42	11 × 44	12 × 44	
16	145.25	170.00	......	......	......	......	......	......
18	114.70	134.00	158.00	173.50	200.00	229.00	242.00	......
20	92.90	108.75	127.75	140.00	162.00	206.00	196.00	......
22	76.85	90.00	105.75	115.50	134.00	153.40	162.25	......
24	64.60	75.65	89.00	97.00	112.75	129.00	136.50	......
26	55.00	64.35	75.65	82.70	96.00	110.00	116.25	......
28	47.40	55.60	65.35	71.40	82.85	94.75	100.25	......
30	41.30	48.40	57.40	62.20	72.15	82.50	87.20	......
32	36.25	42.55	50.00	54.50	63.40	72.50	76.70	......
34	32.15	37.65	44.25	48.40	56.20	64.25	67.90	......
36	28.65	33.60	39.50	43.45	50.00	57.30	60.50	......
38	25.75	30.15	35.50	38.83	45.00	51.50	54.35	......
40	23.25	27.20	32.00	35.00	40.55	46.40	49.00	......
42	21.08	24.70	29.00	31.75	36.80	42.10	44.50	......
44	19.20	22.50	26.43	28.90	33.55	38.40	40.60	......
46	......	20.60	24.20	26.45	30.65	35.10	37.05	......
48	......	18.93	22.25	24.35	28.20	32.25	34.10	......
50	......	......	20.50	22.40	26.00	29.75	31.40	......
52	......	......	......	......	24.00	27.50	29.00	......
54	......	......	......	......	......	25.50	26.90	......

Span.	12 × 46	12 × 48	13 × 48	13 × 50	13 × 52			
20	221.00	251.25	271.50	......	......	......	......	......
22	183.00	207.50	224.50	255.00	286.50	......	......	......
24	153.60	179.90	188.75	215.00	240.75	......	......	......
26	130.75	148.50	160.50	183.00	205.00	......	......	......
28	113.00	128.40	138.50	108.00	177.00	......	......	......
30	98.25	111.90	120.50	137.50	154.00	......	......	......
32	86.30	98.25	106.00	121.00	135.50	......	......	......
34	76.50	87.00	94.00	107.00	120.00	......	......	......
36	68.20	77.50	83.80	95.50	107.00	......	......	......
38	61.20	69.70	75.15	85.75	96.00	......	......	......
40	55.20	62.80	67.75	77.35	86.50	......	......	......
42	50.00	57.00	61.50	70.10	79.75	......	......	......
44	45.60	52.00	56.15	63.90	71.50	......	......	......
46	41.75	47.55	51.30	58.50	65.50	......	......	......
48	38.20	43.65	47.65	53.75	60.20	......	......	......
50	35.35	40.25	43.45	49.55	55.50	......	......	......
52	32.70	37.25	40.15	45.80	51.25	......	......	......
54	30.35	34.50	37.25	42.50	47.55	......	......	......
56	28.15	32.10	34.60	39.50	44.25	......	......	......
58	......	29.95	32.30	36.85	41.25	......	......	......
60	......	......	30.15	34.38	38.50	......	......	......
62	......	......	......	32.25	36.05	......	......	......

Description of Table V.

This table was prepared to serve the same purpose in designing floors, as was Table I in designing beams and girders. All the columns will explain themselves after reading the description of Table I.

The thickness of the floor here given, includes both the top 1-inch wearing surface and 1 inch below the steel tension members, serving for a fire-resisting medium, as well as a finish. In the tables, neither of these thicknesses is considered in obtaining the moment of resistance of the section. The wearing surface was not considered for reasons stated in Part I, and, of course, the protection for the steel could not be, because it lies outside the tension members. In this way, 2 inches is added to the thickness of the floor from which no benefit is expected to resist the moment caused by the loading, and accordingly as thick a concrete floor results as does a wooden one, when figuring for deflection, and a thicker one, when figuring for strength. However, when the thickness of the top wooden floor is added, the thickness of the concrete floor, for resisting strength, will compare favorably with the wooden one. Finally, in the concrete one, we have included the thickness to render the same fire resisting, and, for this reason, have more than offset any objections that might be imposed regarding space occupied.

The moments here given in columns 5 and 6 are figured, allowing a factor of safety of 3.5, which is very ample, especially so when the top 1-inch wearing surface is considered as offering no resistance.

The values of the tensile stress per square inch, given in column 11, brought to bear upon the concrete in the tensile layers, are reduced to a minimum in the case of floors where the ratio of the concrete resisting area to that of the steel, in the tensile layer, is a maximum. This must necessarily be so, since the moment of resistance of the concrete area above the neutral axis, which must withstand the moment of the loading by compression, and which varies as the square of the effective depth of the section is, in the case of floors where the depth is small, very much less for a given resisting area of concrete in the tensile layer than in the case of beams where the effective depth for a like resisting area is large.

For a like reason, because the shearing force resulting from a loading giving a limiting bending moment is, in the case of floors, small in comparison with the resisting area, the stress per square inch produced throughout the section is correspondingly small. Not only this, but the steel tension members are located quite near the neutral axis, and cross the lines of resultant shear at an angle, and hence offer a component to resist the resultant shear. Consequently, it was thought unnecessary to compute tables for shearing values

when failure, with ordinary spans, always results by compression of the upper fibers, allowing of course, that a sufficient tensile moment of resistance has been furnished.

TABLE V.

1	2	3	4	5	6	7	8	9	10	11
Thickness of floor.	Weight per square foot gross.	Area section 12 inches wide.	$\frac{I}{y}$ per foot width.	Moment inch lbs. per foot width.	Moment foot lbs. per foot width.	Distance below center of gravity to neutral axis.	Area of steel per foot width.	Size of steel bars located. 6 inches o. c.	Area of concrete in tensile fiber.	Stress in concrete per square inch.
In.	Lbs.	Sq. in.				In.	Sq. in.		Sq. in.	Lbs.
3.5	43.8	42	4.50	2,250	188	.22	.27	$\frac{3}{8}$	4.22	100
4	50	48	8	4,000	333	.27	.37	$\frac{7}{16}$	3.87	142
4.5	56.2	54	12.8	6,200	517	.25	.38	$\frac{7}{16}$	5.40	104
5	62.5	60	18	9,000	750	.30	.50	$\frac{1}{2}$	5.50	136
5.5	68.7	66	24.5	12,450	1038	.37	.60	$\frac{9}{16}$	6.10	147
6	75	72	32	16,000	1333	.41	.73	$\frac{5}{8}$	6.72	164
6.5	81.2	78	40.5	20,250	1688	.45	.75	$\frac{5}{8}$	6.72	168
7	87.5	84	50	25,000	2083	.47	.82	$\frac{11}{16}$	7.30	168
7.5	94	90	60.5	30,250	2520	.47	.89	$\frac{11}{16}$	7.30	182
8	100	96	72	36,000	3000	.49	.94	$\frac{11}{16}$	7.30	193
8.5	106.2	102	84.5	42,250	3520	.51	1.03	$\frac{3}{4}$	7.87	197
9	112.5	108	98	49,000	4083	.53	1.10	$\frac{3}{4}$	7.87	210
9.5	118.5	114	112.5	56,250	4688	.58	1.18	$\frac{13}{16}$	8.43	210
10	125	120	128	64,000	5333	.58	1.25	$\frac{13}{16}$	8.43	222
10.5	131.5	126	144.5	72,250	6020	.58	1.31	$\frac{13}{16}$	8.43	233
11	137.5	132	162	81,000	6750	.57	1.39	$\frac{7}{8}$	8.98	232
11.5	144	138	181	90,500	7542	.57	1.44	$\frac{7}{8}$	8.98	240
12	150	144	200	100,000	8333	.58	1.51	$\frac{7}{8}$	8.98	252

Description of Table Va.

The amount of steel given in columns 8 and 9 of Table V is needed to furnish the required tensile resistance, to withstand the bending moment, and hence the location of the same should be at right angles to the direction of the supporting beams. Since the section of steel just referred to is stressed to only 16,000 pounds per square inch, but 43 per cent of the section, provided the same had an elastic limit of 37,000 pounds per square inch, would be required, and but 33 per cent of the section, provided a steel with an elastic limit of 50,000 pounds per square inch were used. The remaining 57 or 67 per cent of the area of the steel corresponding to factors of safety of 2.4 and 3 respectively, could be used to overcome any tension caused by an increase of temperature producing expansion. As stated under Part II, a rise of temperature of 70 degrees F. would be the maximum met with in practice, and, as stated there, to overcome this, requires about .6 square inch of steel with an elastic limit of 50,000 pounds per square inch per square foot of concrete. To show that the 67 per cent of steel section just stated is sufficient to take the tension caused by the expansion produced by the rise of 70 degrees, let us take the first case given in Table V. Here .93 square inch of steel per square foot section of concrete is given; 67 per cent of this is .62 square inch. Hence this case is on the safe side, and it is easy to see that all other cases are still safer.

Accordingly, the distribution of steel here given is ample to care for the tension caused by both the loading and the expansion *in one direction* produced by a rise of 70 degrees F. To care for the tension caused by a like expansion in the opposite direction, the following table, giving the sections of steel to resist different temperature changes, is inserted.

TABLE V*a*.

Steel Required for the Following Temperature Changes (Placed Parallel with the Beams).

1	2	3	4	5	6	7	8	9	10	11
	(30° F.)		(40° F.)		(50° F.)		(60° F.)		(70° F.)	
Thickness of floor.	Square inch metal per lineal foot of floor.	Rods 12″ o. c.	Square inch metal per lineal foot of floor.	Rods 12″ o. c.	Square inch metal per lineal foot of floor.	Rods 12″ o. c.	Square inch metal per lineal foot of floor.	Rods 12″ o. c.	Square inch metal per lineal foot of floor.	Rods 12″ o. c.
In.										
3.5	.075	$\frac{5}{16}$	.10	$\frac{3}{8}$	.125	$\frac{3}{8}$	.15	$\frac{7}{16}$	.175	$\frac{7}{16}$
4	.086	$\frac{5}{16}$	.115	$\frac{3}{8}$	.143	$\frac{3}{8}$	.171	$\frac{7}{16}$	.20	$\frac{1}{2}$
4.5	.097	$\frac{5}{16}$	.129	$\frac{3}{8}$	.161	$\frac{7}{16}$	.193	$\frac{7}{16}$	.225	$\frac{1}{2}$
5	.107	$\frac{3}{8}$	.143	$\frac{3}{8}$	.179	$\frac{7}{16}$	.214	$\frac{1}{2}$	.25	$\frac{1}{2}$
5.5	.118	$\frac{3}{8}$	.157	$\frac{7}{16}$	.197	$\frac{1}{2}$	.235	$\frac{1}{2}$	.275	$\frac{9}{16}$
6	.129	$\frac{3}{8}$	.172	$\frac{7}{16}$	.214	$\frac{1}{2}$	.257	$\frac{1}{2}$	.30	$\frac{9}{16}$
6.5	.139	$\frac{3}{8}$	.186	$\frac{7}{16}$	.232	$\frac{1}{2}$	.279	$\frac{9}{16}$	.325	$\frac{5}{8}$
7	.15	$\frac{7}{16}$	.20	$\frac{1}{2}$	.25	$\frac{1}{2}$	.30	$\frac{9}{16}$	.35	$\frac{5}{8}$
7.5	.161	$\frac{7}{16}$	.214	$\frac{1}{2}$	.268	$\frac{9}{16}$	.321	$\frac{5}{8}$	.375	$\frac{5}{8}$
8	.172	$\frac{7}{16}$	.229	$\frac{1}{2}$	.286	$\frac{9}{16}$	.343	$\frac{5}{8}$	.40	$\frac{11}{16}$
8.5	.183	$\frac{7}{16}$	.243	$\frac{1}{2}$	.304	$\frac{9}{16}$	.364	$\frac{5}{8}$	.425	$\frac{11}{16}$
9	.193	$\frac{7}{16}$	.257	$\frac{9}{16}$	.322	$\frac{5}{8}$	.386	$\frac{5}{8}$	.45	$\frac{11}{16}$
9.5	.204	$\frac{1}{2}$	.272	$\frac{9}{16}$	.340	$\frac{5}{8}$	.407	$\frac{11}{16}$	.475	$\frac{11}{16}$
10.	.215	$\frac{1}{2}$	.286	$\frac{9}{16}$	.358	$\frac{5}{8}$	.428	$\frac{11}{16}$	.50	$\frac{3}{4}$
10.5	.226	$\frac{1}{2}$	.300	$\frac{9}{16}$	.376	$\frac{5}{8}$	.45	$\frac{11}{16}$	.525	$\frac{3}{4}$
11	.236	$\frac{1}{2}$	.314	$\frac{9}{16}$	.394	$\frac{5}{8}$	.471	$\frac{11}{16}$	.55	$\frac{3}{4}$
11.5	.247	$\frac{1}{2}$	.329	$\frac{5}{8}$	.412	$\frac{11}{16}$	.492	$\frac{3}{4}$	.575	$\frac{13}{16}$
12	.257	$\frac{1}{2}$	.344	$\frac{5}{8}$	.428	$\frac{11}{16}$	.514	$\frac{3}{4}$	.60	$\frac{13}{16}$

Description of Table VI.

This table serves, in the design of floors, as does Table II in the design of beams and girders. When the span, given in column 2, and the net loading per square foot given under column 4, are known, the corresponding thickness of floor may be obtained from column 1. By the net loading per square foot is meant the live loading. Column 3 gives the gross loading per square foot which includes both the live and the dead loading due to the weight of the floor itself.

Column 5 gives the deflection in inches due to the gross loading and the span. By referring to a series of tests upon the deflection of floors, found in Part II, it will be observed that, while the modulus of elasticity of the girders averaged about 3,000,000, that of the floors at the center of the span, averaged only about 1,600,000. With this in view, column 5 was figured, calling the modulus of elasticity 1,500,000, which is on the safe side.

Under column 6 is given the amount of $\frac{1}{800}$ of the span in inches, with which the deflection under column 5 may be compared. Under the description of Table IV, reasons were given why a deflection of $\frac{1}{800}$ of the span is allowable in the case of beams or girders. Considering this, and in cases where a very level floor is desired, the writer has underscored the longest span for each size, giving a deflection within the limit. In cases where a strictly level floor is not absolutely neces-

sary, the limit of $\frac{1}{800}$ of the span may be exceeded, and surely, by the excess in deflection for all the spans here figured, for the reason that, in figuring the deflection under column 5, the modulus of elasticity was fixed at but half the value assigned to it when a deflection of $\frac{1}{800}$ of the span began to cause cracks. Accordingly, we should not expect cracks to appear until the deflection approached the value $\frac{1}{400}$ of the span, which is borne out by the tests just referred to in Part II.

It will doubtless appear that for a given thickness of floor, and with the ordinary live loads met with in practice, that the limiting span here given is very small. This is so, first, because in the design here given, no attempt has been made to help out the moment of resistance of the concrete in compression by reinforcement. On the other hand, this has been taken as a basis of strength, and enough steel has been inserted to make the tensile moment of resistance equal to it. By inserting steel members into the compression side of the neutral axis and at the same time into the tension side to balance up, the same thickness of floor may be designed for much longer spans, but this comes under special designs, which are not to be considered here. The limit of this reinforcement to increase the moments of resistance of compression and of tension, is fixed by the tensile stress per square inch coming upon the layer of concrete between the steel members in the tensile

layer, which must be kept under 1,500 pounds, as explained in the description of Table I. The second reason is, as stated in the description of Table V, that 2 inches is added to the thickness of the floor from which no benefit as regards strength is figured.

TABLE VI. — *Floors.*

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
3.5	2.5	240	196	.032	.038
	3.0	167	123	.046	.045
	—				
	3.5	123	79	.063	.053
	4.0	94	50	.083	.060
	4.5	74	30	.104	.068
4.0	2.5	425	375	.024	.038
	3.0	296	246	.035	.045
	3.5	218	168	.048	.053
	4.0	167	117	.063	.060
	—				
	4.5	131	81	.079	.068
	5.0	106	56	.098	.075
	5.5	89	39	.115	.082
4.5	2.5	792	735	.024	.038
	3.0	459	401	.029	.045
	3.5	338	281	.040	.053
	4.0	258	201	.052	.060
	4.5	205	148	.066	.068
	5.0	165	108	.081	.075
	—				
	5.5	137	80	.099	[illegible]
	6.0	115	60	.118	[illegible]

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
5.0	2.5	960	896	.017	.038
	3.0	667	904	.025	.045
	3.5	492	429	.032	.053
	4.0	375	308	.044	.060
	4.5	288	224	.055	.068
	5.0	240	177	.069	.075
	5.5	198	134	.084	.082
	6.0	167	104	.101	.090
	6.5	141	78	.117	.098
	7.0	122	59	.136	.105
5.5	2.5	1330	1261	.015	.038
	3.0	923	854	.022	.045
	3.5	680	611	.029	.053
	4.0	519	450	.038	.060
	4.5	411	342	.049	.068
	5.0	332	263	.060	.075
	5.5	275	206	.072	.082
	6.0	231	162	.086	.090
	6 5	197	128	.101	.098
	7.0	169	100	.117	.105
	7.5	148	79	.135	.113
	8.0	130	61	.154	.120
	8.5	115	46	.174	.127
	9.0	102	31	.193	.135
6.0	3.0	1185	1110	.019	.045
	3.5	873	798	.026	.053
	4.0	667	592	.038	.060
	4.5	528	453	.047	.068
	5.0	427	252	.052	.075
	5.5	354	279	.063	.082
	6.0	293	218	.074	.090
	6.5	253	178	.088	.098

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
6.0	7.0	218	133	.102	.105
	7.5	190	115	.117	.112
	8.0	167	92	.133	.120
	8.5	148	73	.149	.127
	9.0	132	57	.169	.135
	9.5	118	43	.188	.144
	10.0	107	32	.209	.150
6.5	3.5	1105	1024	.023	.053
	4.0	845	764	.030	.060
	4.5	669	588	.038	.068
	5.0	540	460	.046	.075
	5.5	449	368	.056	.082
	6.0	376	297	.067	.090
	6.5	322	241	.079	.098
	7.0	275	194	.090	.105
	7.5	241	160	.105	.113
	8.0	211	130	.118	.120
	8.5	188	107	.135	.127
	9.0	166	85	.150	.135
	9.5	150	69	.168	.143
	10.0	135	59	.186	.150
	10.5	123	42	.205	.158
	11.0	112	31	.225	.165
7.0	4.0	1042	955	.027	.060
	4.5	824	937	.034	.068
	5.0	674	587	.043	.075
	5.5	553	466	.051	.082
	6.0	463	376	.061	.090
	6.5	395	308	.071	.098
	7.0	341	254	.083	.105
	7.5	297	210	.096	.113
	8.0	260	173	.108	.120

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
7.0	8.5	231	144	.122	.128
	9.0	207	120	.138	.135
	9 5	186	99	.155	.143
	10.0	166	79	.168	.150
	10.5	151	64	.186	.158
	11.0	138	51	.204	165
	11.5	126	39	.223	.173
	12.0	116	29	.243	.180
7.5	4.0	1260	1166	.025	.060
	4.5	996	902	.031	.068
	5.0	806	712	.044	.075
	5.5	667	573	.047	.082
	6.0	560	466	.056	.090
	6.5	478	384	.066	.098
	7.0	411	317	.076	.105
	7.5	359	265	.087	.113
	8.0	315	221	.100	.120
	8.5	280	186	.113	.128
	9.0	250	156	.127	.135
	9.5	224	130	.142	.143
	10.0	202	108	.156	.150
	10.5	193	99	.171	.158
	11.0	167	73	.189	.165
	11.5	153	59	.206	.173
	12.0	140	46	.224	.180
	12.5	129	35	.243	.198
8.0	4.5	1186	1087	.029	.068
	5.0	960	860	.036	.075
	5.5	797	697	.043	.083
	6.0	667	567	.052	.090
	6.5	570	470	.061	.098
	7.0	490	390	.070	.105

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
8.0	7.5	427	327	.080	.113
	8.0	375	275	.092	.120
	8.5	334	234	.104	.128
	9.0	296	196	.116	.135
	9.5	267	167	.130	.143
	10.0	240	140	.144	.150
	10.5	218	118	.158	.158
	11.0	198	98	.174	.165
	11.5	182	82	.190	.173
	12.0	167	67	.206	.180
	12.5	154	54	.225	.188
	13.0	142	42	.242	.195
	13.5	131	31	.260	.203
8.5	5.0	1126	1020	.033	.075
	5.5	933	827	.040	.083
	6.0	783	677	.048	.090
	6.5	667	561	.056	.098
	7.0	575	469	.065	.105
	7.5	502	396	.075	.113
	8.0	440	334	.085	.120
	8.5	391	285	.097	.128
	9.0	348	242	.108	.135
	9.5	313	207	.120	.143
	10.0	282	176	.133	.150
	10.5	255	149	.146	.158
	11.0	233	127	.161	.165
	11.5	212	106	.175	.173
	12.0	195	89	.191	.180
	12.5	181	75	.209	.188
	13.0	167	61	.226	.195
	13.5	155	49	.243	.203
	14.0	144	38	.262	.210

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
9.0	6.0	906	793	.045	.090
	6.5	773	660	.053	.098
	7.0	666	553	.061	.105
	7.5	582	469	.070	.113
	8.0	510	397	.080	.120
	8.5	454	341	.091	.128
	9.0	404	291	.101	.135
	9.5	362	249	.112	.143
	10.0	326	213	.124	.150
	10.5	296	183	.137	.158
	11.0	270	157	.150	.165
	11.5	248	135	.165	.173
	12.0	227	114	.179	.180
	12.5	209	96	.195	.188
	13.0	193	80	.210	.195
	13.5	179	66	.226	.203
	14.0	162	49	.237	.210
9.5	6.5	890	771	.049	.098
	7.0	766	647	.057	.105
	7.5	670	553	.066	.113
	8.0	586	467	.074	.120
	8.5	521	402	.084	.128
	9.0	463	342	.094	.135
	9.5	417	298	.105	.143
	10.0	375	256	.116	.150
	10.5	341	222	.128	.158
	11.0	310	191	.140	.165
	11.5	284	165	.154	.173
	12.0	261	142	.168	.180
	12.5	240	121	.182	.188
	13.0	222	103	.197	.195
	13.5	207	88	.213	.203
	14.0	192	73	.228	.210
	14.5	179	60	.245	.218
	15.0	165	46	.259	.225

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
10.0	7.0	870	745	.053	.105
	7.5	760	635	.061	.113
	8.0	669	534	.070	.120
	8.5	592	467	.079	.128
	9.0	527	402	.089	.135
	9.5	473	348	.099	.143
	10.0	426	301	.109	.150
	10.5	388	263	.121	.158
	11.0	353	228	.132	.165
	11.5	323	198	.144	.173
	12.0	296	171	.156	.180
	12.5	273	148	.170	.188
	13.0	252	127	.189	.195
	13.5	235	110	.200	.202
	14.0	218	93	.215	.210
	14.5	204	79	.231	.218
	15.0	190	65	.246	.225
	15.5	178	53	.263	.233
10.5	7.5	857	726	.058	.113
	8.0	753	622	.066	.120
	8.5	670	539	.075	.128
	9.0	595	432	.084	.135
	9.5	534	403	.094	.143
	10.0	482	351	.104	.150
	10.5	438	307	.115	.158
	11.0	398	267	.125	.165
	11.5	364	233	.136	.173
	12.0	335	204	.149	.180
	12.5	309	178	.162	.188
	13.0	285	154	.175	.195
	13.5	264	133	.188	.203
	14.0	246	115	.203	.210
	14.5	229	98	.218	.218
	15.0	215	84	.234	.225

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
10.5	15.5	201	70	.249	.233
	16.0	188	57	.264	.240
11.0	8.0	780	642	.058	.120
	8.5	692	554	.066	.128
	9.0	618	480	.074	.135
	9.5	555	417	.083	.143
	10.0	500	362	.091	.150
	10.5	453	315	.101	.158
	11.0	415	277	.111	.165
	11.5	378	240	.121	.173
	12.0	348	210	.132	.180
	12.5	320	182	.143	.188
	13.0	295	157	.154	.195
	13.5	275	137	.167	.203
	14.0	255	117	.179	.210
	14.5	238	100	.193	.218
	15.0	223	85	.207	.225
	15.5	208	70	.220	.233
	16.0	196	58	.234	.240
	16.5	184	46	.249	.248
11.5	7.0	1233	1089	.046	.105
	7.5	1085	940	.054	.113
	8.0	940	795	.060	.120
	8.5	838	693	.068	.128
	9.0	746	602	.076	.135
	9.5	670	526	.085	.143
	10.0	604	459	.094	.150
	10.5	549	405	.104	.158
	11.0	500	356	.114	.165
	11.5	458	314	.125	.173
	12.0	420	276	.135	.180
	12.5	387	240	.148	.188
	13.0	357	213	.159	.195

TABLE VI. — *Floors.* — Continued.

1	2	3	4	5	6
Thickness of floor.	Span.	Load square foot gross.	Load square foot net.	Deflection caused by loading.	$\frac{1}{800}$ Span.
Inches.	Feet.	Lbs.	Lbs.	Inches.	Inches.
11.5	13.5	322	188	.167	.203
	14.0	309	165	.185	.210
	14.5	288	144	.198	.218
	15.0	268	124	.212	.225
	15.5	251	107	.226	.233
	16.0	236	91	.241	.240
	16.5	222	78	.257	.248
	17.0	210	66	.273	.255
	17.5	197	53	.288	.263
12.0	7.5	1186	1036	.049	.113
	8.0	1042	892	.057	.120
	8.5	925	775	.064	.128
	9.0	823	673	.072	.135
	9.5	740	589	.080	.143
	10.0	666	516	.088	.150
	10.5	604	454	.097	.158
	11.0	552	402	.107	.165
	11.5	504	354	.116	.173
	12.0	463	313	.127	.180
	12.5	427	279	.138	.188
	13.0	395	245	.150	.195
	13.5	366	216	.161	.203
	14.0	340	190	.173	.210
	14.5	318	168	.180	.218
	15.0	304	154	.204	.225
	15.5	278	128	.212	.233
	16.0	260	110	.225	.240
	16.5	245	95	.240	.248
	17.0	230	80	.254	.255
	17.5	218	68	.270	.263
	18.0	206	56	.286	.270

Description of Table VII.

This table gives, for different sizes of columns, both with circular and with octagonal sections, the corresponding safe total load in tons that the same will carry, paying due regard to the ratio of height to diameter by caring for eccentricity of loading. In determining the sizes, the concrete alone was figured to withstand by compression the total loading, while the steel inserted within the concrete, was designed to resist all stress both of tension and of compression caused by bending, due to any eccentricity of loading as stated below. The portion of load, limiting the amount of eccentricity, was fixed at one quarter of the total loading, and the amount cared for was that due to this loading acting about a leverage of an amount equal to the effective radius of the column. By the effective radius is meant the distance from the center of the column to the center of the steel bars.

Circular and octagonal shapes were figured because they allow for the most effective arrangement of the rods, and besides, octagonal shapes are very successfully formed. It is also designed to use eight rods in each case, spaced equally apart around the column, and at a distance from the center of the column equal to 1 inch less than the radius. Eight rods thus placed give the greatest moment of resistance about any axis for a given amount of material, and hence are the most economical.

TABLE VII. — *Columns.*

Circular section.						Octagonal section.			
1	2	3	4	5	6	7	8	9	10
Size.	Area section.	Weight per foot.	Safe load.	Height of column.	Size of rods. (Use 8.)	Weight per foot.	Safe load.	Height of column.	Size of rods.
In.	Sq. in.	Lbs.	Tons.	Feet.	In.	Lbs.	Tons.	Feet.	In.
5	19.6	20	8.3	4–6	$\frac{3}{16}$	18	7.5	4–6	$\frac{3}{16}$
				7–11	$\frac{1}{4}$			7–11	$\frac{1}{4}$
				12	$\frac{5}{16}$			12	$\frac{5}{16}$
6	28.0	29	11.9	5	$\frac{3}{16}$	26	10.3	5–6	$\frac{3}{16}$
				6–9	$\frac{1}{4}$			7–10	$\frac{1}{4}$
				10–15	$\frac{5}{16}$			11–15	$\frac{5}{16}$
7	38.5	40	16.4	6–7	$\frac{1}{4}$	36	14.8	6–8	$\frac{1}{4}$
				8–12	$\frac{5}{16}$			9–13	$\frac{5}{16}$
				13–17	$\frac{3}{8}$			14–17	$\frac{3}{8}$
8	50.3	52	21.4	7–10	$\frac{5}{16}$	47	19.3	7	$\frac{1}{4}$
				11–15	$\frac{3}{8}$			8–12	$\frac{5}{16}$
				16–20	$\frac{7}{16}$			13–17	$\frac{3}{8}$
								18–20	$\frac{7}{16}$
9	63.6	66	27	8–9	$\frac{5}{16}$	60	24.4	8–10	$\frac{5}{16}$
				10–13	$\frac{3}{8}$			11–15	$\frac{3}{8}$
				14–19	$\frac{7}{16}$			16–20	$\frac{7}{16}$
				20	$\frac{1}{2}$				
10	78.5	82	33.4	9–12	$\frac{3}{8}$	74	30.0	9	$\frac{5}{16}$
				13–17	$\frac{7}{16}$			10–14	$\frac{3}{8}$
				18–20	$\frac{1}{2}$			15–19	$\frac{7}{16}$
								20	$\frac{1}{2}$
11	95.0	99	40.4	8–11	$\frac{3}{8}$	89	36.5	8	$\frac{5}{16}$
				12–15	$\frac{7}{16}$			10–12	$\frac{3}{8}$
				16–20	$\frac{1}{2}$			13–17	$\frac{7}{16}$
								18–20	$\frac{1}{2}$
12	113.0	118	48.0	8–10	$\frac{3}{8}$	106	43.3	8–11	$\frac{3}{8}$
				11–14	$\frac{7}{16}$			12–15	$\frac{7}{16}$
				15–18	$\frac{1}{2}$			16–20	$\frac{1}{2}$
				19–20	$\frac{9}{16}$				

TABLE VII. — *Columns.* — Continued.

Circular section.						Octagonal section.			
1	2	3	4	5	6	7	8	9	10
Size.	Area section.	Weight per foot.	Safe load.	Height of column.	Size of rods. (Use 8.)	Weight per foot.	Safe load.	Height of column.	Size of rods.
In.	Sq. in.	Lbs.	Tons.	Feet.	In.	Lbs.	Tons.	Feet.	In.
13	132.7	138	56.5	8–9	$\frac{3}{8}$	125	51.0	8–10	$\frac{3}{8}$
				10–13	$\frac{7}{16}$			11–14	$\frac{7}{16}$
				14–17	$\frac{1}{2}$			15–19	$\frac{1}{2}$
				18–20	$\frac{9}{16}$			20	$\frac{9}{16}$
14	153.9	160	65.5	8–9	$\frac{3}{8}$	144	59.0	8–10	$\frac{3}{8}$
				10–12	$\frac{7}{16}$			11–13	$\frac{7}{16}$
				13–16	$\frac{1}{2}$			14–18	$\frac{1}{2}$
				17–20	$\frac{9}{16}$			19–20	$\frac{9}{16}$
15	176.7	184	75.2	8–11	$\frac{7}{16}$	166	67.8	8	$\frac{3}{8}$
				12–15	$\frac{1}{2}$			9–12	$\frac{7}{16}$
				16–18	$\frac{9}{16}$			13–16	$\frac{1}{2}$
				19–20	$\frac{5}{8}$			17–20	$\frac{9}{16}$
16	201.0	210	85.5	8–10	$\frac{7}{16}$	189	77.0	8	$\frac{3}{8}$
				11–14	$\frac{1}{2}$			9–11	$\frac{7}{16}$
				15–17	$\frac{9}{16}$			12–15	$\frac{1}{2}$
				18–20	$\frac{5}{8}$			16–19	$\frac{9}{16}$
								20	$\frac{5}{8}$
17	227.0	236	96.5	8–10	$\frac{7}{16}$	213	87.0	8	$\frac{3}{8}$
				11–13	$\frac{1}{2}$			9–11	$\frac{7}{16}$
				14–16	$\frac{9}{16}$			12–14	$\frac{1}{2}$
				17–20	$\frac{5}{8}$			15–18	$\frac{9}{16}$
								19–20	$\frac{5}{8}$
18	254.5	255	108.0	8–9	$\frac{7}{16}$	239	97.5	8–10	$\frac{7}{16}$
				10–12	$\frac{1}{2}$			11–13	$\frac{1}{2}$
				13–15	$\frac{9}{16}$			14–17	$\frac{9}{16}$
				16–19	$\frac{5}{8}$			18–20	$\frac{5}{8}$
				20	$\frac{11}{16}$				
19	283.5	295	120.5	8–9	$\frac{7}{16}$	266	109.0	8–9	$\frac{7}{16}$
				10–11	$\frac{1}{2}$			10–13	$\frac{1}{2}$
				12–14	$\frac{9}{16}$			14–16	$\frac{9}{16}$
				15–18	$\frac{5}{8}$			17–20	$\frac{5}{8}$
				19–20	$\frac{11}{16}$				

TABLE VII. — *Columns.* — Continued.

Circular section.						Octagonal section.			
1	2	3	4	5	6	7	8	9	10
Size.	Area section.	Weight per foot.	Safe load.	Height of column.	Size of rods. (Use 8).	Weight per foot.	Safe load.	Height of column.	Size of rods.
In.	Sq. in.	Lbs.	Tons.	Feet.	In.	Lbs.	Tons.	Feet.	In.
20	314.1	327	133.3	8	$\frac{7}{16}$	295	120.0	8–9	$\frac{7}{16}$
				9–11	$\frac{1}{2}$			10–12	$\frac{1}{2}$
				12–14	$\frac{9}{16}$			13–15	$\frac{9}{16}$
				15–17	$\frac{5}{8}$			16–19	$\frac{5}{8}$
				18–20	$\frac{11}{16}$			20	$\frac{11}{16}$
22	380	396	161.7	8–10	$\frac{1}{2}$	358	145.0	8	$\frac{7}{16}$
				11–12	$\frac{9}{16}$			9–11	$\frac{1}{2}$
				13–15	$\frac{5}{8}$			12–14	$\frac{9}{16}$
				16–19	$\frac{11}{16}$			15–17	$\frac{5}{8}$
				20	$\frac{3}{4}$			18–20	$\frac{11}{16}$
24	452	472	192.4	8–9	$\frac{1}{2}$	427	173.5	8–10	$\frac{1}{2}$
				10–11	$\frac{9}{16}$			11–12	$\frac{9}{16}$
				12–14	$\frac{5}{8}$			13–15	$\frac{5}{8}$
				15–17	$\frac{11}{16}$			16–19	$\frac{11}{16}$
				18–20	$\frac{3}{4}$			20	$\frac{3}{4}$
26	531	553	225.5	8	$\frac{1}{2}$	500	203.2	8–9	$\frac{1}{2}$
				9–10	$\frac{9}{16}$			10–11	$\frac{9}{16}$
				11–12	$\frac{5}{8}$			12–14	$\frac{5}{8}$
				13–16	$\frac{11}{16}$			15–18	$\frac{11}{16}$
				17–19	$\frac{3}{4}$			19–20	$\frac{3}{4}$
				20	$\frac{13}{16}$				
28	616	641	262.0	8	$\frac{1}{2}$	578	235.0	8	$\frac{1}{2}$
				9	$\frac{9}{16}$			9–11	$\frac{9}{16}$
				10–12	$\frac{5}{8}$			12–13	$\frac{5}{8}$
				13–15	$\frac{11}{16}$			14–16	$\frac{11}{16}$
				16–18	$\frac{3}{4}$			17–20	$\frac{3}{4}$
				19–20	$\frac{13}{16}$				
30	707	735	300.0	8–9	$\frac{9}{16}$	662	270.0	8	$\frac{1}{2}$
				10–11	$\frac{5}{8}$			9–12	$\frac{9}{16}$
				12–14	$\frac{11}{16}$			13–15	$\frac{5}{8}$
				15–16	$\frac{3}{4}$			16–18	$\frac{11}{16}$
				17–19	$\frac{13}{16}$			19–20	$\frac{3}{4}$
				20	$\frac{7}{8}$				

Description of Table VIII.

Table VIII is drawn up both to facilitate making estimates, and to show at a glance the comparative costs, for equal strength, of the three kinds of construction given — namely, reinforced concrete, structural steel, and slow-burning, when used in the design of floors in the shape of beams or girders.

The basis of the cost of the concrete given under column 3 is from data taken by the writer upon actual work, and represents fair working conditions. It includes all temporary false work, and everything to make a finished piece of work, and even allows going over the exposed surface with a cement wash after pointing up and removing irregularities where necessary. The cost of the steel used in connection with the concrete, is based upon the price f.o.b of 2 cents per pound, to which is added another 2 cents per pound for handling, cutting to lengths, placing in forms, and wiring to place where necessary.

The cost of the structural shapes given under column 7 is based upon a price of 2.5 cents per pound f.o.b, to which is added $10 per ton, or $\frac{1}{2}$ cent per pound to cover the cost of placing, bolting, or riveting to place, and painting, which is little enough.

The cost of wooden beams is figured upon a basis of a price of $35 per thousand feet upon the site for planed stock, which is increased by $10

per thousand feet to cover the cost of sizing, placing, and fitting, but no other finish. The cost of the slow-burning construction, as figured here, applies principally to northern sections of the country, and should be considerably reduced to meet southern conditions.

TABLE VIII. — *Comparative Costs. — Beams for Equal Strength.*

1	2	3	4	5	6	7	8	9	10
	REINFORCED CONCRETE.					STEEL.		WOOD.	
Moment.	Size concrete beam.	Cost of concrete per lineal foot.	Cost of steel per lineal foot.	Total cost per foot length.	Size steel beam.	Cost per foot length. (Plain.)	Cost per foot length. (Fire. proofed.)	Size wood beam Southern Pine.	Cost per foot length.
In.-lbs.	Inches.							Inches.	
25,250	2.5×10	$0.08	$0.08	$0.16	3″– 6.5 lbs.	$0.19	$0.28	4×6	$0.09
37,800	2.5× 2	.09	.09	.18	4″– 7.5	.22	.32	4×8	.12
45,500	3×12	.11	.10	.21	4″– 7.5	.22	.32	3×10	.12
63,500	3×14	.13	.12	.25	5″– 9.75	.29	.40	4×10	.15
84,500	4×14	.17	.15	.32	5″–12.25	.37	.48	4×12	.18
112,500	4×16	.20	.17	.37	6″–12.25	.37	.49	6×10	.23
140,500	5×16	.25	.24	.49	7″–15	.45	.57	6×12	.27
180,000	5×18	.28	.27	.55	7″–17	.53	.65	6×14	.32
225,500	5×20	.31	.30	.61	8″–18	.54	.67	8×14	.36
270,500	6×20	.38	.34	.72	8″–25.5	.76	.89	8×14	.42
332,500	6×22	.41	.34	.75	9″–30	.90	1.03	8×16	.48
398,500	6×24	.45	.42	.87	10″–30	.90	1.03	10×16	.60
464,500	7×24	.53	.46	.99	12″–31.5	.95	1.09	12×16	.72
547,500	7×26	.57	.46	1.03	12″–31.5	.95	1.09	14×16	.84

639,500	7×28	.61	.52	1.13	12″- 40	1.20	1.34	14×18	.95
725,000	8×28	.70	.58	1.28	15″- 42	1.26	1.41	14×18	.95
840,000	8×30	.75	.64	1.39	15″- 42	1.26	1.41	14×20	1.05
962,000	8×32	.80	.70	1.50	15″- 45	1.35	1.50	16×18	1.08
1,082,500	9×32	.90	.77	1.67	15″- 55	1.65	1.80	16×20	1.20
1,227,500	9×34	.95	.84	1.79	18″- 55	1.65	1.82	16×22	1.32
1,380,000	9×36	1.01	.84	1.85	18″- 55	1.65	1.82	16×24	1.44
1,532,000	10×36	1.13	1.00	2.13	20″- 65	1.95	2.13	....	....
1,702,000	10×38	1.19	1.00	2.19	20″- 65	1.95	2.13	....	....
1,905,000	10×40	1.25	1.08	2.33	20″- 70	2.10	2.28	....	....
2,092,000	11×40	1.37	1.16	2.53	20″- 80	2.40	2.58	....	....
2,312,500	11×42	1.45	1.25	2.70	20″- 80	2.40	2.58	....	....
2,540,000	11×44	1.52	1.25	2.77	24″- 80	2.43	2.61	....	....
2,683,000	12×44	1.65	1.34	2.99	24″- 80	2.43	2.61	....	....
2,900,000	12×46	1.73	1.44	3.17	24″- 90	2.70	2.89	....	....
3,175,000	12×48	1.80	1.44	3.24	24″-100	3.00	3.19	....	....

Description of Table IX.

The purpose of this table is to give a comparative cost, for equal strength, of a fire resisting, reinforced concrete floor in comparison with the ordinary floor used in the slow-burning construction.

The cost of the concrete given under column 3 is based upon actual results under ordinary conditions, but differs in the price per unit of volume from that of beams and girders. The cost of steel given under column 4 is figured upon the same basis as was explained in Table VIII. The item, "Cost of Troweling," includes the cost of labor both in applying the 1-inch finish, and the screeding and troweling same to a finished surface.

For wooden floors, the price here given is based upon the cost of spruce plank laid at $35 per thousand feet; of southern pine laid at $45; of 1-inch No. 2 maple top flooring laid and dressed at $60; and No. 1 maple at $80 per thousand feet.

TABLE IX. — *Comparative Costs. — Floors for Equal Strength.*

1	2	3	4	5	6	7	8	9	10	11
	Reinforced concrete.					Wood.				
Safe moment in inch-lbs. per foot width.	Thickness of floor.	Cost of concrete per sq. ft.	Cost of steel per sq. ft.	Cost of troweling per sq. ft.	Total cost per sq. ft.	Spruce or H.P. plank size.	Cost of same per sq. ft.	Cost per sq. ft. maple No. 2 top floor.	Total cost No. 2 top floor.	Total cost No. 1 top floor.
	In.					Spruce				
2,250	3.5	$0.13	$0.04	$0.03	$0.20	2″	$0.07	$0.06	$0.13	$0.15
4,000	4.0	.15	.05	.03	.23	2″	.07	.06	.13	.15
6,200	4.5	.17	.05	.03	.25	2″	.07	.06	.13	.15
9,000	5.0	.19	.07	.03	.29	3″	.11	.06	.17	.19
12,450	5.5	.21	.10	.03	.33	3″	.11	.06	.17	.19
16,000	6.0	.23	.11	.03	.37	4″	.14	.06	.20	.22
20,250	6.5	.24	.11	.03	.38	4″	.14	.06	.20	.22
25,000	7.0	.26	.13	.03	.42	4″	.14	.06	.20	.22
						H. P.				
30,250	7.5	.28	.13	.03	.44	4″	.18	.06	.24	.26
36,000	8.0	.30	.13	.03	.46	5″	.23	.06	.29	.31
42,250	8.5	.32	.15	.03	.50	5″	.23	.06	.29	.31
49,000	9.0	.34	.15	.03	.52	5″	.23	.06	.29	.31
56,250	9.5	.36	.18	.03	.57	6″	.27	.06	.33	.35
64,000	10.0	.38	.18	.03	.59	6″	.27	.06	.33	.35
72,250	10.5	.39	.18	.03	.60	6″	.27	.06	.33	.35

Description of Table X.

Table X is similar in all respects to Table IX, but compares the two kinds of constructions upon a fairer basis — that is, the relative costs for like stiffness or for like deflections. Since reinforced concrete, because of a high modulus of elasticity, gives a stiffer floor than does the wood, this basis of comparison as regards cost, more nearly shows up the concrete floor in its proper merits in this particular sphere.

Table X. — *Comparative Costs. — Floors for Equal Deflection.*

1	2	3	4	5	6	7
	Reinforced concrete.			Wood.		
Safe moment in inch-pounds per foot width.	Thickness of floor.	Actual resisting depth.	Cost per sq. ft. complete.	Thickness of spruce or H. P. plank. (Inches.)	Total cost per sq. ft. with No. 2 maple top floor.	Total cost per sq. ft. with No. 1 maple top floor.
	Inches.	Inches.		Spruce.		
2,250	3.5	1.5	$0.20	2″	$0.13	$0.17
4,000	4.0	2.0	.23	3″	.17	.19
6,200	4.5	2.5	.25	4″	.20	.22
				H. P.		
9,000	5.0	3.0	.29	4″	.24	.26
12,400	5.5	3.5	.33	5″	.29	.31
16,000	6.0	4.0	.37	5″	.29	.31
20,250	6.5	4.5	.38	6″	.33	.35

Description of Table XI.

The following table is given both to assist in estimating the cost of reinforced concrete columns, and to serve as a means of comparison in cost between the more general forms of construction — namely, cast iron, steel or wrought iron, and wood.

As before, the cost given here for reinforced concrete is taken from average results of actual construction of columns. The basis of computing the cost of the other kinds of columns is taken: for cast iron, 2.5 cents per pound, erected in the building; for steel, 3 cents per pound for plain shapes, and 3.5 cents for riveted sections erected; and for wood $45 per thousand feet in place.

TABLE XI.

Concrete (circular section).						Cast iron.		Steel.		Wood.	
1	2	3	4	5	6	7	8	9	10	11	12
Safe load.	Diameter of column.	Height of column.	Cost of concrete per foot height of column.	Cost of steel per foot height of column.	Total cost per foot height of column.	Size of column. (Diameter × thickness.)	Cost per foot height.	Area section.	Cost per foot height.	Size of square column.	Cost per foot height.
Tons.	Inches.	Feet.				Inches.		Sq. in.		Inches.	
8.3	5	4–6	$0.08	$0.04	$0.12	....	....	....	....	4	$ 0.06
		7–11		.07	.15	....	....	....	....	5	.10
		12		.10	.18	....	....	3.3	$0.34	7	.18
11.9	6	5	.12	.04	.16	....	....	....	....	5	.10
		6–9		.07	.19	....	....	....	....	5	.10
		10–15		.10	.22	....	....	4.8	.49	6–9	.14–.30
16.4	7	6–7	.16	.07	.23	....	....	....	....	6	.14
		8–12		.10	.26	....	....	....	....	6	.14
		13–17		.16	.32	....	....	6.6	.66	7–10	.18–.38
21.4	8	7–10	.21	.10	.31	....	....	....	....	6	.14
		11–15		.16	.37	6 × $\frac{3}{4}$	$1.10	....	....	7–9	.18–.30
		16–20		.21	.42	....	....	8.6	.87	10–12	.38–.54

27.0	9	8–9	.26	.10	.36	....	....	....	....	7	.18
		10–13		.16	.42	....	....	....	....	7–8	.18–.24
		14–19		.21	.47	$6\times\frac{3}{4}$	1.10	....	....	9–11	.30–.45
		20		.27	.53	$8\times\frac{3}{4}$	1.58	10.8	1.08	12	.54
33.4	10	9–12	.33	.16	.49	$6\times\frac{3}{4}$	1.10	....	....	8	.24
		13–17		.21	.54	$7\times\frac{3}{4}$	1.28	....	....	9–10	.30–.38
		18–20		.27	.60	$8\times\frac{3}{4}$	1.58	13.4	1.35	11–12	.45–.54
40.4	11	8–11	.40	.16	.56	$6\times\frac{3}{4}$	1.10	....	....	9	.30
		12–15		.21	.61	$7\times\frac{3}{4}$	1.28	....	....	9	.30
		16–20		.27	.67	$8\times\frac{3}{4}$	1.58	16.2	1.64	10–12	.38–.54
48.0	12	8–10	.47	.16	.63	$7\times\frac{3}{4}$	1.28	....	....	10	.38
		11–14		.21	.68	$8\times\frac{3}{4}$	1.58	....	....	10	.38
		15–18		.27	.74	8×1	1.95	19.2	2.26	11	.45
		19–20		.33	.80	8×1	1.95	....	....	12	.54
56.5	13	8–9	.55	.16	.71	$7\times\frac{7}{8}$	1.50	....	....	11	.45
		10–13		.21	.76	$8\times\frac{7}{8}$	1.72	....	....	11	.45
		14–17		.27	.82	8×1	1.95	....	....	11	.45
		18–20		.33	.88	9×1	2.20	22.6	2.66	12	.54
65.5	14	8–9	.64	.16	.80	$8\times\frac{7}{8}$	1.72	....	....	....	....
		10–12		.21	.85	8×1	1.95	....	....	....	....

TABLE XI. — Continued.

Concrete (circular section).						Cast iron.		Steel.		Wood.	
1	2	3	4	5	6	7	8	9	10	11	12
Safe load.	Diameter of column.	Height of column.	Cost of concrete per foot height of column.	Cost of steel per foot height of column.	Total cost per foot height of column.	Size of column. (Diameter × thickness.)	Cost per foot height.	Area section.	Cost per foot height.	Size of square column.	Cost per foot height.
Tons.	Inches.	Feet.				Inches.		Sq. in.		Inches.	
		13–16		$0.27	$0.91	9×1	$2.20	....	....	....	....
		17–20		.33	.97	10× $\frac{7}{8}$	2.14	26.2	$3.09	12	$0.54
75.2	15	8–11	$0.74	.21	.95	9× $\frac{7}{8}$	2.08	....	....	....	....
		12–15		.27	1.01	9×1$\frac{1}{8}$	2.48	....	....	....	....
		16–18		.33	1.07	10×1	2.45	....	....	....	....
		19–20		.42	1.16	10×1$\frac{1}{8}$	2.76	30.1	3.54	12	.54
85.5	16	8–10	.84	.21	1.05	9× $\frac{7}{8}$	2.08	....	....	....	....
		11–14		.27	1:11	10× $\frac{7}{8}$	2.14	....	....	....	....
		15–17		.33	1.17	10×1$\frac{1}{8}$	2.76	....	....	....	....
		18–20		.42	1.26	10×1$\frac{1}{4}$	3.06	34.2	4.02	13	.63
96.5	17	8–10	.94	.21	1.15	9×1	2.20	....	....	....	....
		11–13		.27	1.21	10×1	2.45	....	....	....	....

		14–16		.33	1.27	$10 \times 1\frac{1}{8}$	2.76	....	....	....	....
		17–20		.42	1.36	$11 \times 1\frac{1}{8}$	3.03	38.6	4.55	14	.74
108.0	18	8–9	1.02	.21	1.23	$9 \times 1\frac{1}{8}$	2.48	....	....	....	....
		10–12		.27	1.29	$10 \times 1\frac{1}{8}$	2.76	....	....	....	....
		13–15		.33	1.35	$10 \times 1\frac{1}{4}$	3.06	....	....	....	....
		16–19		.42	1.44	$11 \times 1\frac{1}{4}$	3.38	....	....	....	....
		20		.50	1.52	$11 \times 1\frac{1}{4}$	3.38	43.2	5.07	15	.84
120.5	19	8–9	1.18	.21	1.39	$10 \times 1\frac{1}{8}$	2.76	....	....	....	....
		10–11		.27	1.45	$10 \times 1\frac{1}{4}$	3.06	....	....	....	....
		12–14		.33	1.51	$11 \times 1\frac{1}{8}$	3.03	....	....	....	....
		15–18		.42	1.60	$11 \times 1\frac{3}{8}$	3.71	....	....	....	....
		19–20		.50	1.68	$12 \times 1\frac{1}{4}$	3.68	48.2	5.66	16	.96
133.3	20	8	1.31	.21	1.52	$10 \times 1\frac{1}{8}$	2.76	....	....	....	....
		9–11		.27	1.58	$11 \times 1\frac{1}{8}$	3.03	....	....	....	....
		12–14		.33	1.64	$11 \times 1\frac{1}{4}$	3.38	....	....	....	....
		15–17		.42	1.73	$12 \times 1\frac{1}{4}$	3.68	....	....	....	....
		18–20		.50	1.81	$12 \times 1\frac{3}{8}$	4.05	53.3	6.25	17	1.09
161.7	22	8–10	1.52	.27	1.79	$11 \times 1\frac{1}{4}$	3.39	....	....	....	....
		11–12		.33	1.85	$11 \times 1\frac{3}{8}$	3.71	....	....	....	....
		13–15		.42	1.94	$12 \times 1\frac{1}{4}$	3.68	....	....	....	....
		16–19		.50	2.02	$12 \times 1\frac{3}{8}$	4.05	64.7	7.60	....	....
		20		.61	2.13	$12 \times 1\frac{3}{8}$	4.05	....	....	....	....

TABLE XI. — Continued.

Concrete (circular section).						Cast iron.		Steel.		Wood.	
1	2	3	4	5	6	7	8	9	10	11	12
Safe load.	Diameter of column.	Height of column.	Cost of concrete per foot height of column.	Cost of steel per foot height of column.	Total cost per foot height of column.	Size of column. (Diameter × thickness.)	Cost per foot height.	Area section.	Cost per foot height.	Size of square column.	Cost per foot height.
Tons.	Inches.	Feet.				Inches.		Sq. in.		Inches.	
192.4	24	8–9	$1.81	$0.27	$2.08	12×1¼	$3.68	....	....	....	....
		10–11		.33	2.14	12×1⅜	4.05	....	....	....	....
		12–14		.42	2.23	13×1⅜	4.38	....	....	....	....
		15–17		.50	2.31	13×1½	4.77	....	....	....	....
		18–20		.61	2.42	14×1½	5.15	77	$ 9.05	....	....
225.5	26	8	2.31	.27	2.58	13×1⅜	4.38	....	....	....	....
		9–10		.33	2.64	13×1½	4.77	....	....	....	....
		11–12		.42	2.73	13×1½	4.77	....	....	....	....
		13–16		.50	2.81	14×1½	5.15	....	....	....	....
		17–19		.61	2.92	14×1⅝	5.55	....	....	....	....
		20		.72	3.03	15×1½	5.52	90	10.60	....	....

262	28	8	2.56	.27	2.83	$14\times1\frac{1}{2}$	5.15	....	....	....	....
		9		.33	2.89	$14\times1\frac{1}{2}$	5.15	....	....	....	....
		10–12		.42	2.98	$14\times1\frac{5}{8}$	5.55	....	....	....	....
		13–15		.50	3.06	$15\times1\frac{5}{8}$	5.97	....	....	....	....
		16–18		.61	3.17	$15\times1\frac{3}{4}$	6.45	....	....	....	....
		19–20		.72	3.28	$16\times1\frac{5}{8}$	6.37	105	12.30	....	....
300	30	8–9	2.94	.33	3.27	$14\times1\frac{5}{8}$	5.55	....	....	....	....
		10–11		.42	3.36	$15\times1\frac{5}{8}$	6.70	....	....	....	....
		12–14		.50	3.94	$15\times1\frac{3}{4}$	6.45	....	....	....	....
		15–16		.61	3.55	$16\times1\frac{3}{4}$	6.85	....	....	....	....
		17–19		.72	3.66	$16\times1\frac{3}{4}$	6.85	....	....	....	....
		20		.83	3.77	$18\times1\frac{5}{8}$	7.17	120	14.10	....	....

TABLE XI. — Continued.

CONCRETE — OCTAGONAL SECTION.

1	2	3	4	5	6
Safe load.	Circumscribed diameter of column.	Height of column.	Cost of concrete per foot height of column.	Cost of steel per foot height of column.	Total cost per foot height of column.
Tons.	Inches.	Feet.			
7.5	5	4–6	$0.07	$0.04	$0.11
		7–11		.07	.14
		12		.10	.17
10.3	6	5–6	.10	.04	.14
		7–10		.07	.17
		11–15		.10	.20
14.8	7	6–8	.14	.07	.21
		9–13		.10	.24
		14–17		.16	.30
19.3	8	7	.19	.07	.26
		8–12		.10	.29
		13–17		.16	.35
		18–20		.21	.40
24.4	9	8–10	.24	.10	.34
		11–15		.16	.40
		16–20		.21	.45
30.0	10	9	.30	.10	.40
		10–14		.16	.46
		15–19		.21	.51
		20		.27	.57
36.5	11	8	.36	.10	.46
		10–12		.16	.52
		13–17		.21	.57
		18–20		.27	.63
43.3	12	8–11	.42	.16	.58
		12–15		.21	.63
		16–20		.27	.69

TABLE XI. — Continued.

CONCRETE — OCTAGONAL SECTION.

1	2	3	4	5	6
Safe load.	Circum-scribed diameter of column.	Height of column.	Cost of concrete per foot height of column.	Cost of steel per foot height of column.	Total cost per foot height of column.
Tons.	Inches.	Feet.			
51.0	13	8–10	$0.50	$0.16	$0.66
		11–14		.21	.71
		15–19		.27	.77
		20		.33	.83
59.0	14	8–10	.58	.16	.74
		11–13		.21	.79
		14–18		.27	.85
		19–20		.33	.91
67.8	15	8	.66	.16	.82
		9–12		.21	.87
		13–16		.27	.93
		17–20		.33	.99
77.0	16	8	.76	.16	.92
		9–11		.21	.97
		12–15		.27	1.03
		16–19		.33	1.09
		20		.42	1.18
87.0	17	8	.85	.16	1.01
		9–11		.21	1.06
		12–14		.27	1.12
		15–18		.33	1.18
		19–20		.42	1.27
97.5	18	8–10	.96	.21	1.17
		11–13		.27	1.23
		14–17		.33	1.29
		18–20		.42	1.38
109.0	19	8–9	1.06	.21	1.27
		10–13		.27	1.33
		14–16		.33	1.39
		17–20		.42	1.48

TABLE XI. — Continued.

CONCRETE — OCTAGONAL SECTION.

1	2	3	4	5	6
Safe load.	Circum-scribed diameter of column.	Height of column.	Cost of concrete per foot height of column.	Cost of steel per foot height of column.	Total cost per foot height of column.
Tons.	Inches.	Feet.			
120.0	20	8–9	$1.18	$0.21	$1.39
		10–12		.27	1.45
		13–15		.33	1.51
		16–19		.42	1.60
		20		.50	1.68
145.0	22	8	1.43	.21	1.64
		9–11		.27	1.70
		12–14		.33	1.76
		15–17		.42	1.85
		18–20		.50	1.93
173.5	24	8–10	1.71	.27	1.98
		11–12		.33	2.04
		13–15		.42	2.13
		16–19		.50	2.21
		20		.61	2.32
203.2	26	8–9	2.00	.27	2.27
		10–11		.33	2.33
		12–14		.42	2.42
		15–18		.50	2.50
		19–20		.61	2.61
235.0	28	8	2.31	.27	2.58
		9–11		.33	2.64
		12–13		.42	2.73
		14–16		.50	2.81
		17–20		.61	2.92
270.0	30	8	2.64	.27	2.91
		9–12		.33	2.97
		13–15		.42	3.06
		16–18		.50	3.14
		19–20		.61	3.25

TABLE XII. — *Amounts of Cement, Sand, and Stone Required for Concrete Mixtures of Various Proportions.*

CONCRETE WITH $2\frac{1}{2}$-INCH STONE.

Proportions of mixture.			Required for 1 cubic yard.			
Cement.	Sand.	Stone.	Cement.	Sand.	Stone.	Stone.
			Barrels.	Cu. yards.	Cu. yards.	Tons.
1	1	2.0	2.72	0.41	0.83	1.31
1	1	2.5	2.41	0.37	0.92	1.40
1	1	3.0	2.16	0.33	0.98	1.49
1	1.5	2.5	2.16	0.49	0.82	1.14
1	1.5	3.0	1.96	0.45	0.89	1.22
1	1.5	3.5	1.79	0.41	0.96	1.31
1	1.5	4.0	1.64	0.38	1.00	1.38
1	2.0	3.0	1.78	0.54	0.81	1.02
1	2.0	3.5	1.66	0.50	0.88	1.11
1	2.0	4.0	1.53	0.47	0.93	1.18
1	2.0	4.5	1.43	0.43	0.98	1.28
1	2.5	3.5	1.51	0.58	0.81	.93
1	2.5	4.0	1.42	0.54	0.87	1.02
1	2.5	4.5	1.33	0.51	0.91	1.09
1	2.5	5.0	1.26	0.48	0.96	1.15
1	2.5	5.5	1.18	0.44	0.99	1.20
1	3.0	4.0	1.32	0.60	0.80	.89
1	3.0	4.5	1.24	0.57	0.85	.95
1	3.0	5.0	1.17	0.54	0.89	1.03
1	3.0	5.5	1.11	0.51	0.93	1.09
1	3.0	6.0	1.06	0.48	0.97	1.15
1	3.5	5.0	1.11	0.59	0.85	.91
1	3.5	5.5	1.06	0.56	0.89	.98
1	3.5	6.0	1.00	0.53	0.92	1.04
1	3.5	6.5	0.96	0.51	0.95	1.09
1	3.5	7.0	0.91	0.49	0.98	1.14

TABLE XII. — *Amounts of Cement, Sand, and Stone.* — Cont'd.

CONCRETE WITH STONE $\frac{3}{4}$-INCH AND UNDER.

Proportions of mixture.			Required for 1 cubic yard.			
Cement.	Sand.	Stone.	Cement.	Sand.	Stone.	Stone.
			Barrels.	Cu. yards.	Cu. yards.	Tons.
1	1	2.5	2.10	0.32	0.80	1.51
1	1	3.0	1.89	0.29	0.86	1.58
1	1	3.5	1.71	0.26	0.91	1.64
1	1	4.0	1.55	0.24	0.94	1.70
1	1.5	3.0	1.71	0.39	0.78	1.36
1	1.5	3.5	1.57	0.36	0.83	1.42
1	1.5	4.0	1.46	0.33	0.88	1.49
1	1.5	4.5	1.34	0.31	0.91	1.53
1	1.5	5.0	1.24	0.28	0.94	1.55
1	2.0	3.5	1.44	0.44	0.77	1.25
1	2.0	4.0	1.34	0.41	0.81	1.31
1	2.0	4.5	1.26	0.38	0.86	1.38
1	2.0	5.0	1.17	0.36	0.89	1.42
1	2.0	6.0	1.03	0.31	0.94	1.53
1	2.5	4.0	1.24	0.47	0.75	1.18
1	2.5	4.5	1.16	0.44	0.80	1.24
1	2.5	5.0	1.10	0.42	0.83	1.29
1	2.5	5.5	1.03	0.39	0.86	1.36
1	2.5	6.0	0.98	0.37	0.89	1.40
1	2.5	7.0	0.88	0.33	0.93	1.44
1	3.0	5.0	1.03	0.47	0.78	1.11
1	3.0	5.5	0.97	0.44	0.81	1.24
1	3.0	6.0	0.92	0.42	0.84	1.29
1	3.0	6.5	0.88	0.40	0.87	1.35
1	3.0	7.0	0.84	0.38	0.89	1.38
1	3.0	7.5	0.80	0.37	0.91	1.40
1	3.0	8.0	0.76	0.35	0.93	1.44
1	3.5	6.0	0.88	0.46	0.80	1.20
1	3.5	6.5	0.83	0.44	0.82	1.25
1	3.5	7.0	0.80	0.43	0.85	1.26
1	3.5	7.5	0.76	0.41	0.87	1.31
1	3.5	8.0	0.73	0.39	0.89	1.36
1	3.5	8.5	0.71	0.38	0.91	1.37
1	3.5	9.0	0.68	0.36	0.92	1.42

DESCRIPTION OF TABLE XIII.

This table has for a purpose to compare various proportions of mixture or mixes as regards strength. For a basis by which all other mixes are compared, a 1–1.5–3 mix was taken and called 100 per cent efficient.

Column 2 considers that the strength of the mix depends upon the amount of cement, and that the effect of the cement is positive in regard to affecting strength. Hence, all other mixes are given in percentages of strength, calling the 1–1.5–3 mix unity, determined as just stated.

Column 3 considers that the strength of the mix varies as the absence of the aggregate, and that the effect of the aggregate is negative as affecting strength. Accordingly, all other mixes are given in percentages of strength, considering the 1–1.5–3 mix the basis.

Table XIII. — *Relative Strength of Different Proportions of Mixture.*

1	2	3
Proportion of mixture.	Deduced by amount of cement.	Deduced by amount of aggregate
1—1.5—3.0	1.00	1.00
1—1.5—3.5	.92	.90
1—1.5—4.0	.85	.82
1—2—3.5	.85	.82
1—2—4.0	.78	.75
1—2—4.5	.74	.69
1—2.5—4.0	.73	.69
1—2.5—4.5	.68	.64
1—2.5—5.0	.64	.60
1—3—5.0	.60	.56
1—3—5.5	.57	.53
1—3—6.0	.54	.50
1—3.5—6.0	52	.48
1—3.5—6.5	.49	.45
1—3.5—7.0	.47	.43

PART IV.

DESIGNS OF REINFORCED CONCRETE TRUSSES.

TRUSSED ROOFS.

The growing demand for boiler houses, forge shops, foundries, dye houses, and such classes of buildings that will successfully withstand deterioration or destruction from the effects of gases, vapors, or fire, warrants a brief chapter on the design, cost, and construction details of reinforced concrete trussed roofs. Up to the present time this kind of construction is the only one that can be conscientiously recommended to successfully resist the effects of the agencies mentioned above. Let it not be understood that the classes of roofs named before are the only ones to which this system is applicable. To the contrary, no limitation can be placed on the scope of its use in cases where final cost, permanency, and low insurance rates are to be considered.

No doubt an important reason why its use has been looked upon with such distrust and ill-favor, is because of the fussy construction details which add considerably, and often with restriction, to the first cost, by way of form work, temporary shoring, and inconvenience to handling materials, thereby impeding progress, and prohibiting economy. Much of this difficulty can be lessened or overcome by adhering to some of the details hereafter mentioned.

Truss Skeleton.

First of all, a brief outline is given of truss members that may be successfully and economically molded outside of their ultimate locations, or in other words, apart from the truss. These pieces may be formed exactly to dimensions, or may be fitted with bolts or any necessary future iron work, and this with every assurance that when erected in place, such bolts or iron work will be properly spaced for the part it has to play in the building. Secondly, these pieces being formed on or near terra firma, can receive all due attention when pouring. Thirdly, all shoring or bracing which would be necessary were the piece molded in place, can be done away with, since the piece is self-supporting, after having set a reasonable length of time, and hence a saving. Fourthly, pieces thus molded are, or should be, free from sag, which so often occurs when monolithically poured, and are therefore free from eccentric loading, and no part overstressed through negligence of inspection. Fifthly, such members can be erected as inexpensively as can wooden ones, and can be supported on the forms which mold the monolithic parts without other shoring than would be necessary for said molds.

The principal parts that can be treated thus are the diagonal braces, which are always in compression when in place. The design of these may be successfully treated along the following lines.

First of all, these braces should, if rectangular or square in section, contain four rods or bars, one in or near every corner, and these of such size as to render a sufficient moment of resistance about the central axis that offers the least radius of gyration, to enable the piece to be raised into place with the lashing at any point along its length without causing cracks. If circular or octagonal in section, use eight rods. Secondly, the length of the braces should be 2 or 3 inches greater than the neat distance required between chords, thus allowing a tie of 1 or $1\frac{1}{2}$ inches into each chord. In addition to this tie, the four rods previously mentioned should project 4 to 8 inches beyond either end of the brace, and these projections, when the surrounding parts are poured, serve to form a monolithic mass with the whole. These braces may be designed to carry pillow blocks, supporting shafting, by molding in a web between the brace and its adjoining vertical tie, into which web may be anchored hook bolts for receiving the pillow blocks. In a similar manner, webs may be formed to support piping, or to accommodate any features required. Again when purlin braces are required, either to decrease the section of the purlin, or to longitudinally brace the lower chords of the main trusses, these may be successfully formed by outside molding. As it often becomes convenient to hang shafting parallel to the axes of the main trusses, such braces seem especially adapted for receiving it by casting webs

between them and the vertical tie members of the main trusses. These braces should be designed as stated under diagonal braces.

All other parts of the roof frame, comprising the upper and lower chords of the main trusses, all vertical ties, as well as all purlins and rafters, should be formed *in situ.*

In the following tables, giving the designs of various trusses to meet certain requirements, it will be noted that two sizes have been given for both the upper and lower chords. The sizes marked "Reinforcement Sizes" should be used merely to refer back to similar sizes under "Beam and Girder Designs" in Part III, to pick out the steel sections required to withstand the bending moment brought about, in the case of upper chords, by a uniformly distributed dead and live loading, including their own weights, over the particular span. In the case of lower chords, the reinforcement is merely to withstand the bending moment due to their own weights uniformly distributed over the spans designated. After paying due attention to continuous girder effects, as treated in Part III, and equally applicable here, no other reinforcement except that to resist shear is required in these parts.

Purlins and rafters should be treated as beams, the reinforcement for the same being found by referring the sizes given to corresponding ones in Part III, bearing in mind the effect of continuity.

In the tables following, the areas of steel sections

are given for the truss rods or bars in the vertical tension members. In treating this, the most important feature of the design, may call forth various opinions. It is suggested from a practical standpoint, to use round rods with threaded ends and nuts, having flat plate bearing washers at each end. These plates, to develop by compression of the concrete, the safe tensile stress in the rod, should have a net bearing area on the concrete twenty times the area of the rod. The upper bearing plate should be bent to conform to the *A* shape of the upper chord, should be located as near the upper surface as practicable, and above a layer of rods, the total area of which, combined with the concrete and steel section below, will care for half the shearing stress developed by the safe tension in the truss rod. Table III, Part III, gives safe shearing forces under three conditions, which the net section of chord under the bearing plate will resist with no other reinforcement than that required in the chord to withstand bending. When further reinforcement is required, it may be designed in accordance with the data given in Table III, Part III, and this supplied as just stated, below the bearing plate. Again, should this reinforcement just determined, prove inadequate to care for the tensile stress in the upper fibers over the rods, as required by the tables on "Beams having Fixed Ends," it (the reinforcement) should be further increased. Needless to say, the bearing area of the upper bent plate pre-

viously referred to, should be the horizontally projected area. The lower bearing plate should, of course, be located below the steel, in the underside of the lower chord. The steel section here, if the occasion requires, should be increased as stated before, so that the safe shearing force either side of the truss rod will develop half the safe tensile stress of the rod. Previous to pouring the lower chord, the truss rods with their lower bearing plates should be supported in place, and later molded monolithically with the lower chord. The truss rods should then be surrounded with a form of square section having bevelled corners, and of a size equal to the width of the lower chord. Just as soon as the concrete in the lower chord has set sufficiently to resist the static head, due to the mass of plastic concrete above, these forms should be filled in around the truss rod with cement, and the truss completed. In the four corners of these vertical tension members, should be placed rods of light section to prevent shrinkage cracks between these members and the upper and lower chords. Consequently, they should protrude far enough into the chords to obtain the adhesion necessary to develop their safe tensile stress, or be anchored. The combined area of these rods should be such as to resist by tension the contraction due to the evaporation of water from the vertical members. This will vary, depending upon the plasticity of the concrete when poured, and may be somewhat overcome by using

a rather dry concrete well rammed in place, and by keeping the surface moistened with water after removing the forms so as to keep it from setting faster than the thicker masses in the chords. In the absence of experiments, this part of the design can be carried out by approximation only, but the approximation can be kept within bounds when we know the percentage of free water in the concrete, over that required by chemical action, to the whole mass, since the shrinkage is in direct proportion to this percentage, and especially when the precautions before named are regarded.

The upper bearing plate should not be put on until the concrete has reached the level in the upper chord to receive the rods under the plate. After these latter have been placed and properly covered with a stiff cement, the plate may be inserted over the end of the rod and screwed down firmly, and bedded by means of the nut above. Needless to say, at the apex of the truss, the rods under the plate should be bent to suit the chord, and also the bearing plates. The vertical members at each panel point should be similarly treated.

As before mentioned, due regard must be paid to continuity over panel points. Accordingly, the tension thus caused by bending, should be cared for by rods in the upper side of both chords at these points.

Particularly should the shear caused on either side of the panel points in both chords be treated

with no little concern, as these are the weakest points in the design if not properly considered, both in the design and in the workmanship. Accordingly, at all such points, longitudinal shear bars should be inserted on either side of the panel points to care for the longitudinal shear along the neutral axis as well as the varying longitudinal shear between the neutral axis and the upper and lower layers, as treated in Part III.

In cases where there is but one span of trusses over the width of the building, and the ends of these rest on the outside walls, in order to prevent the tendency of the upper chord to shear by the lower chord or produce an excessive thrust on the outside walls, it is suggested to insert a rod fastened at one end around the first truss or panel point rod from the support, the other end to be threaded and supplied with a nut and bearing plate. This plate should have an area equal to the difference in areas between the concrete and steel sizes given for the lower chord, and should be located over the wall at a point to receive the thrust from the upper chord. The size of the rod just mentioned should be one-twentieth that of the bearing plate, and its location should be at the center of the lower chord. This provision is not necessary at a bearing where trusses extend in both directions.

Roof (Proper).

In order to minimize the excessive dead load of a roof that has to withstand a given loading, Table XIV has been drawn up for such special usage.

For light roof frames, and where light live loads may be figured on, wooden plank, either of hard pine, or white pine, are practicable and economical. For white pine may be substituted Northern or Canadian pine, fir, or spruce. To render the exposed surface fire-resisting, a cement plaster, of varying thickness, is applied to the underside of the plank. This plaster is held on by metal lath or expanded metal fastened directly to the plank, or to furring strips on the plank. Any danger considered imminent from dry rot may be obviated by supplying a sufficient number of small vent holes through the plaster. To surely guard against decay from the above source, it is well to kyanize the planks before putting them in place. Kyanizing is especially adapted to spruce lumber, and will serve as a double precaution against rot, should moisture in any way get through the plaster. The plank may be held down by spiking them into a dovetailed nailing piece let into the upper chord flush with its upper surface.

The cost of concrete roofs may be lessened to some extent by casting them in slabs of a length equal to the bay, and of a width suitable for handling, and raising into place. These may be

fastened to the upper chord by clips driven into a dovetailed wooden nailing piece as mentioned above. Each slab should be fastened down before the adjoining slab is placed. To produce the best results, each slab should be so formed as to break all joints at right angles to the trusses or bearings by halving the lap of one onto that of the other. When molding, clips should be left projecting from the underside of the slabs and anchored around the lower reinforcement in the slabs. Before raising to place, a very light metal lath, expanded metal or latticed wire, should be attached to the underside of the slab by means of the clips already mentioned. This serves as a key to hold on a plaster coat of cement, which should be added in one thin coat over the entire underside surface of the roof to give a desired finish and to protect the metal fastenings. As a poor, but economical substitute, the plaster finish may be omitted, and the slabs bedded in cement mortar at the bearings and lapped joints, afterwards cutting a V-joint between each slab with a chisel and tool. This will reduce the cost given in Table XIV*a* about ten cents per square foot.

Description of Table XIV.

Table XIV is drawn up to give special roofs to meet special requirements. To obtain the greatest strength for the least weight of material has been the particular object sought.

Line 1 gives the gross loading per square foot to carry which the roofs have been designed. Line 2 gives the net loading which equals the live loading after the dead weight of the roof itself has been deducted. The remainder of the table will explain itself.

TABLE XIV.

	8-Foot Span.				
1. Gross load, square foot. (Lbs.)	50	75	100	75	100
2. Net load, square foot. (Lbs.)	25	37.5	50	50	75
3. Thickness plank. (In.)	2.5 W.P.	.....	.....	2.5 W.P.	3 W.P.
4. Thickness plaster. (In.)	1.5	.....	.....	1.5	1¼
5. Thickness concrete. (In.)	....	3	4	....	....
6. Steel in comp. Square inch per lineal foot.	....	.28	.....	....	....
7. Steel in tension. Square inch per lineal foot.	....	.65	.5	....	....
	10-Foot Span.				
1. Gross load, square foot. (Lbs.)	50	75	100	75	100
2. Net load, square foot. (Lbs.)	25	37.5	50	50	75
3. Thickness plank. (In.)	2.5 W.P.	.....	.....	3 W.P.	3 H.P.
4. Thickness plaster. (In.)	1.5	.....	.....	1¼	1
5. Thickness concrete. (In.)	....	3	4	....	....
6. Steel in comp. Square inch per lineal foot.	....	.64	.4	....	....
7. Steel in tension. Square inch per lineal foot.	....	1.01	.9	....	....

TABLE XIV. — Continued.

	12-Foot Span.				
1. Gross load, square foot. (Lbs.)	50	75	100	75	100
2. Net load, square foot. (Lbs.)	25	37.5	50	50	75
3. Thickness plank. (In.)	3 W.P.	.....	......	3 H.P.	4 H.P.
4. Thickness plaster. (In.)	$1\frac{1}{4}$	.....	......	1	$\frac{3}{4}$
5. Thickness concrete. (In.)	....	3	4	....	....
6. Steel in comp. Square inch per lineal foot.	....	1.08	.91	....	....
7. Steel in tension. Square inch per lineal foot.	....	1.45	1.41	....	....

	14-Foot Span.			
1. Gross load, square foot. (Lbs.)	50	75	100	75
2. Net load, square foot. (Lbs.) ..	25	37.5	50	25
3. Thickness plank. (In.)........	3 H.P.	......	....	4 H.P.
4. Thickness plaster. (In.).......	1	......	....	$\frac{3}{4}$
5. Thickness concrete. (In.)	....	4	4	....
6. Steel in comp. Square inch per lineal foot.	....	1.60	1.36	....
7. Steel in tension. Square inch per lineal foot.	....	1.97	1.86	....

	16-Foot Span.			
1. Gross load, square foot. (Lbs.)	50	75	100	75
2. Net load, square foot. (Lbs.) ..	25	37.5	50	50
3. Thickness plank. (In.)........	4 H.P.	......	....	4 H.P.
4. Thickness plaster. (In.).......	$\frac{3}{4}$	......	....	$\frac{3}{4}$
5. Thickness concrete. (In.)	....	3	4	....
6. Steel in comp. Square inch per lineal foot.	....	2.28	1.90	....
7. Steel in tension. Square inch per lineal foot.	....	2.65	2.40	....

TABLE XIV. — Continued.

	18- and 20-Foot Span.				
1. Gross load, square foot. (Lbs.)	50	75	100	75	100
2. Net load, square foot. (Lbs.)	25	37.5	50	50	75
3. Thickness plank. (In.)	2.5 W.P.	.....	.....	3 W.P.	5 H.P.
4. Thickness plaster. (In.)	1.5	.....	.....	$1\frac{1}{4}$	1
5. Thickness concrete. (In.)	....	3	4	....	....
6. Steel in comp. Square inch per lineal foot.	....	.64	.4	....	....
7. Steel in tension. Square inch per lineal foot.	....	1.01	.9	....	....

NOTE. — Spans mentioned above as 18 and 20 feet are actually reduced to 9 and 10 feet, respectively, by means of the intermediate rafters, and are mentioned as such only to afford a ready reference when using Table XVII. Likewise, bays of 24 and 30 feet actually give spans for the roofing of but 8 and 10 feet, respectively.

DESCRIPTION OF TABLE XIV*a*.

This table gives the approximate costs of roofs per square foot of projected area covered, when constructed of the materials and designed as stated in Table XIV. All costs given are in dollars or fractional parts thereof.

The span mentioned under Column 1 is just a reference back to Table XIV, where may be found the amounts of steel for the various spans here specified.

These costs are fair averages of ordinary cases. In cases where the average working conditions cannot be obtained, allowance must be made.

For cases where wooden plank are used, two sets of costs are given in the table. The first is for plain lumber; the second is for the same stock, but kyanized.

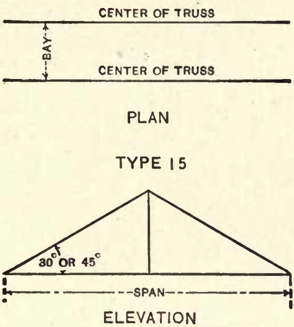
TABLE XIV*a*.— *Cost in Dollars.*

1	2	3	4	5	6	
	Formed in place.	Slabs set in place.	Thickness of plaster.			
			¾″	1″	1¼″	1½″
Thickness of Concrete:						
3″ (8 ft. span)	$0.40	$0.38	......	......	......	......
3″ (10 ft. span)....	.45	.40	......	......	......	......
3″ (12 ft. span)....	.52	.47	......	......	......	......
3″ (14 ft. span)....	.60	.53	......	......	......	......
3″ (16 ft. span)....	.72	.63	......	......	......	......
Thickness of Concrete:						
4″ (8 ft. span).....	.45	.39	......	......	......	......
4″ (10 ft. span)....	.53	.45	......	......	......	......
4″ (12 ft. span)....	.61	.52	......	......	......	......
4″ (14 ft. span)....	.68	.58	......	......	......	......
4″ (16 ft. span)....	.79	.67	......	......	......	......
Thickness of Plank:						
2½″ W.P.	....	....	......	......	......	$0.34
						.36
3 ″ W.P.	......	....	......	......	$0.36	......
					.39	......
3 ″ W.P.	....	....	......	$0.32	......	......
				.35	......	......
4 ″ W.P.	......	....	$0.35	......	......	......
			.39	.....	......	......

DESCRIPTION OF TABLE XV.

This table is drawn up to furnish complete designs of trusses, of the type here shown in the sketch for various bays and spans.

Columns 2, 3, and 4 give sizes of upper chords when the slope of the truss is 45 degrees. The figures in the upper set are sizes from which the reinforcement may be determined by reference to



tables in Part III, while the lower set gives the concrete sizes.

Columns 5, 6, and 7 are similar in every way to columns 2, 3, and 4, giving corresponding sizes for 30 degrees slope trusses.

Columns 8 to 10, inclusive, and 11 to 13, inclusive, give sizes of lower chords, and are similar to columns 2 to 4 and 5 to 7, respectively.

Columns 14 to 16 contain the reinforcement sizes from which the steel for the lower chord may be determined by reference to the corresponding sizes in Table I, Part III.

Columns 17 to 19 give areas in square inches of truss rods for the corresponding bays and spans noted.

TABLE XV.

25-Foot Span.

1	2	3	4	5	6	7
	Sizes of Upper Chord.					
Bay (feet).	45° Slope. Load sq.ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
	5×18	6×20	6×22	5×16	5×18	6×20
8	5.5×18	6.5×20	6.5×22	5.5×16	5.5×18	7×20
	5×20	6×22	6×24	5×18	5×20	6×22
10	5.5×20	6.5×22	6.5×24	5.5×18	6×20	7×22
	6×20	6×24	7×26	5×18	6×20	6×24
12	6.5×20	6.5×24	7.5×26	5.5×18	7×20	7×24
	6×22	7×24	7×26	5×20	6×22	6×24
14	6.5×22	7.5×24	8×26	5.5×20	7×22	7×24
	6×22	7×24	7×28	6×20	6×24	7×24
16	6.5×22	8×24	8×28	7×20	7×24	8×24

	Sizes of Lower Chord.					
	8	9	10	11	12	13
8	3.5×10	4×12	4.5×12	4×10	4×12	5×12
10	3.5×10	4.5×12	6×12	4×10	5×12	6×12
12	4×12	4.5×14	6×14	4×12	5×14	6.5×14
14	4.5×12	6×12	6×16	5×12	6.5×12	7×16
16	4.5×12	6×14	7×16	5×12	7×14	8×16

	Lower Chord.			Truss Rod.		
	14	15	16	17	18	19
	Reinforcement Sizes.			Area in sq. in.		
8	2.5×10	2.5×12	3×12	.20	.24	.30
10	2.5×10	3×12	4×12	.20	.30	.36
12	2.5×12	3×14	4×14	.25	.36	.47
14	3×12	4×12	4×16	.31	.40	.58
16	3×12	4×14	5×16	.31	.51	.66

NOTE. — Upper figures indicate reinforcement sizes. Lower figures indicate concrete sizes.

Table XV. — Continued.

30 Foot Span.

Sizes of Upper Chord.

1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
	6×20	6×24	7×26	5×20	6×20	6×24
8	6.5×20	6.5×24	7.5×26	5.5×20	6.5×20	7×24
	6×22	7×24	7×28	6×20	6×22	7×24
10	6.5×22	7.5×24	7.5×28	6.5×20	7×22	8×24
	6×24	7×26	8×28	6×20	6×24	7×26
12	6.5×24	7.5×26	9×28	6.5×20	7×24	8×26
	6×24	7×28	8×30	6×22	7×24	7×28
14	6.5×24	7.5×28	9×30	7×22	8×24	8×28
	7×26	8×28	8×32	6×24	7×26	8×28
16	7.5×26	9×28	9×32	7×24	8×26	9.5×28

Sizes of Lower Chord.

	8	9	10	11	12	13
8	3.5×12	4×12	4.5×14	4×12	5×12	5.5×14
10	3.5×12	4.5×12	5.5×14	4×12	5.5×12	6.5×14
12	4×12	5.5×12	5.5×16	5×12	7×12	7×16
14	4.5×12	5.5×14	7×16	5×12	7×14	8×16
16	4.5×14	5.5×16	7×18	5×14	7×16	8×18

	Lower Chord.			Truss Rod.		
	14	15	16	17	18	19
	Reinforcement Sizes.			Area in sq. in.		
8	2.5×12	3×12	3×14	.30	.37	.48
10	2.5×12	3×12	4×14	.30	.42	.57
12	3×12	4×12	4×16	.37	.52	.70
14	3×12	4×14	5×16	.40	.61	.79
16	3×14	4×16	5×18	.44	.70	.90

TABLE XV. — Continued.

35-Foot Span.

	Sizes of Upper Chord.					
1	**2**	**3**	**4**	**5**	**6**	**7**
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
	6×24	7×28	8×28	6×20	6×24	7×26
8	6.5×24	7.5×28	8.5×28	6.5×20	7×24	8×26
	7×26	8×28	8×32	6×22	7×26	7×28
10	7.5×26	8.5×28	8.5×32	6.5×22	8×26	8×28
	7×28	8×30	9×32	6×24	7×26	8×28
12	7.5×28	8.5×30	10×32	7×24	8×26	9×28
	7×28	8×32	9×34	7×24	8×28	8×32
14	7.5×28	9×32	10×34	8×24	9×28	9×32
	8×28	9×32	9×36	7×26	8×30	8×32
16	8.5×28	10×32	10×36	8×26	9×30	9.5×32
	Sizes of Lower Chord.					
	8	**9**	**10**	**11**	**12**	**13**
8	4×14	4×14	5.5×14	4.5×14	5×14	6.5×14
10	4×14	5.5×14	5.5×16	4.5×14	6.5×14	6.5×16
12	4×14	5.5×14	7×16	5×14	7×14	8×16
14	4.5×14	5.5×16	7×18	5×14	7×16	8×18
16	5.5×14	7×16	7×20	6.5×14	8×16	8.5×20
	Lower Chord.			Truss Rod.		
	14	**15**	**16**	**17**	**18**	**19**
	Reinforcement Sizes.			Area in sq. in.		
8	3×14	3×14	4×14	.46	.52	.66
10	3×14	4×14	4×16	.46	.66	.76
12	3×14	4×14	5×16	.51	.72	.92
14	3×14	4×16	5×18	.51	.82	1.05
16	4×14	5×16	5×20	.67	.93	1.15

Table XV. — Continued.

40-Foot Span.

1	2	3	4	5	6	7
	Sizes of Upper Chord.					
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
	7×28	8×30	8×32	6×24	7×26	8×28
8	7.5×28	8.5×30	8.5×32	6.5×24	8×26	9×28
	8×28	8×32	9×34	7×24	8×28	8×32
10	8.5×28	8.5×32	10×34	8×24	9×28	9×32
	8×30	9×32	9×36	7×26	8×30	9×32
12	8.5×30	9.5×32	10×36	8×26	9×30	10×32
	8×32	9×34	10×38	7×28	8×32	9×34
14	8.5×32	10×34	11×38	8×28	9×30	10.5×34
	9×32	9×36	10×40	8×28	9×32	9×36
16	9.5×32	10×36	11×40	9×28	10×32	10.5×36

	Sizes of Lower Chord.					
	8	9	10	11	12	13
8	5×16	5.5×16	5.5×16	5.5×16	6×16	6.5×16
10	5×16	5.5×16	6.5×16	5.5×16	6.5×16	8×16
12	5×16	5.5×16	7×16	6×16	7×16	8.5×18
14	5.5×16	7×16	7.5×16	6×16	8×16	8.5×20
16	5.5×16	7×18	8.5×20	6.5×16	8.5×18	10×20

	Lower Chord.			Truss Rod.		
	14	15	16	17	18	19
	Reinforcement Sizes.			Area in sq. in.		
8	4×16	4×16	4×16	.74	.80	.87
10	4×16	4×16	5×16	.74	.87	1.06
12	4×16	4×16	5×18	.80	.94	1.32
14	4×16	5×16	5×20	.80	1.06	1.42
16	4×16	5×18	6×20	.87	1.32	1.66

TABLE XV. — Continued.

45-Foot Span.

1	2	3	4	5	6	7
	Sizes of Upper Chord.					
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
	8×30	9×32	9×36	7×26	8×28	8×32
8	8.5×30	9.5×32	9.5×36	7.5×26	9×28	9×32
	8×32	9×36	10×38	7×28	8×32	9×34
10	8.5×32	9.5×36	11×38	7.5×28	9×32	10×34
	9×32	10×36	10×40	8×28	9×32	9×36
12	9.5×32	11×36	11×40	9×28	10×32	10×36
	9×34	10×38	11×42	8×30	9×34	10×36
14	9.5×34	11×38	12×42	9×30	10×34	10.5×36
	10×36	10×40	11×44	9×32	9×36	10×40
16	10.5×36	11×40	12×44	10×32	10×36	10.5×40

	Sizes of Lower Chord.					
	8	9	10	11	12	13
8	6×18	6×18	6.5×18	6.5×18	7×18	7.5×18
10	6×18	6.5×18	7×18	6.5×18	7.5×18	9×18
12	6×18	6.5×18	7×18	7×18	8×18	8.5×20
14	6.5×18	7×18	7.5×18	7.5×18	8.5×18	10×20
16	6.5×18	7×20	8.5×22	7.5×18	8.5×20	10×22

	Lower Chord.			Truss Rod.		
	14	15	16	17	18	19
	Reinforcement Sizes.			Area in sq. in.		
8	5×18	5×18	5×18	1.10	1.18	1.28
10	5×18	5×18	5×18	1.10	1.28	1.52
12	5×18	5×18	5×20	1.18	1.35	1.59
14	5×18	5×18	6×20	1.28	1.48	1.88
16	5×18	5×20	6×22	1.28	1.59	2.06

TABLE XV. — Continued.

50-Foot Span.

Bay (feet).	Sizes of Upper Chord.					
	2	**3**	**4**	**5**	**6**	**7**
	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
	9×34	10×36	10×40	8×28	8×32	9×34
8	9.5×34	10.5×36	10.5×40	8.5×28	9×32	10×34
	9×36	10×40	11×42	8×30	9×34	10×36
10	9.5×36	10.5×40	12×42	8.5×30	10×34	11×36
	10×36	11×40	11×44	9×32	9×36	10×40
12	10.5×36	11.5×40	12×44	10×32	10×36	11×40
	10×38	11×42	12×44	9×34	10×38	11×42
14	10.5×38	12×42	13×44	10×34	11×38	12.5×42
	10×40	11×44	13×48	10×36	10×40	11×44
16	10.5×40	12×44	14×48	11×36	11×40	12.5×44

Bay (feet).	Sizes of Lower Chord.					
	8	**9**	**10**	**11**	**12**	**13**
8	6×20	6.5×20	6.5×20	6.5×20	7×20	7.5×20
10	6×20	6.5×20	8×20	6.5×20	7.5×20	8×20
12	6×20	6.5×20	8×20	7×20	7.5×20	9.5×20
14	6×20	7×20	8.5×20	7.5×20	8×20	10×22
16	7.5×20	8.5×20	8.5×24	7.5×20	9.5×20	10.5×24

Bay (feet).	Lower Chord.			Truss Rod.		
	14	**15**	**16**	**17**	**18**	**19**
	Reinforcement Sizes.			Area in sq. in.		
8	5×20	5×20	5×20	1.36	1.46	1.56
10	5×20	5×20	5×20	1.36	1.56	1.66
12	5×20	5×20	6×20	1.46	1.56	1.98
14	5×20	5×20	6×22	1.56	1.70	2.28
16	5×20	6×20	6×24	1.56	1.98	2.66

Description of Table XVa.

Under this table is compiled data for estimating the weights of various truss skeletons of the type shown under Table XV, for different bays and spans. The weights given are in pounds per square foot of projected area covered. These weights do not include the weight of the roof proper. Use may be made of this data in designing columns or other supports to receive the trusses, or in estimating the yardage of their volume for cost purposes, since approximately 4,000 pounds of homogeneous concrete equals one cubic yard. After the yardage of concrete is known, the quantities of constituents forming the concrete may be figured by reference to Table XII, Part III, when the proportions of mixture are known.

TABLE XVA.—*Weight of Truss Skeleton per Square Foot of Area Covered.*

25-Foot Span.

1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.)			30° Slope. Load sq. ft. (lbs.)		
	50	75	100	50	75	100
8	23	30	33	19	21	29
10	20	27	31	16	21	26
12	20	25	31	14	20	25
14	19	24	29	14	19	23
16	17	23	28	14	19	23
Average	20	26	30	15	20	25

30-Foot Span.

8	30	36	44	23	28	35
10	25	32	39	21	25	33
12	24	30	39	18	24	31
14	21	28	37	18	24	29
16	22	29	35	17	23	30
Average	24	31	39	19	25	32

35-Foot Span.

8	37	46	54	31	38	49
10	35	43	50	26	40	42
12	31	38	49	25	34	40
14	27	37	45	24	33	39
16	27	37	43	25	32	37
Average	31	40	48	26	35	41

TABLE XVa. — Continued.

40-Foot Span.

1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.)			30° Slope. Load sq. ft. (lbs.)		
	50	75	100	50	75	100
8	49	59	62	36	44	52
10	44	50	61	32	41	48
12	38	45	54	30	37	46
14	35	44	53	27	33	43
16	34	42	52	26	34	41
Average	40	48	56	30	38	46

45-Foot Span.

8	61	71	79	45	54	61
10	52	63	75	38	49	58
12	47	59	65	36	45	51
14	43	54	63	33	41	47
16	43	50	61	33	38	46
Average	49	59	69	37	45	52

50-Foot Span.

8	75	87	95	53	62	72
10	63	76	90	44	56	64
12	57	68	79	44	49	61
14	51	64	73	40	49	61
16	49	60	76	39	45	58
Average	59	71	83	44	52	63

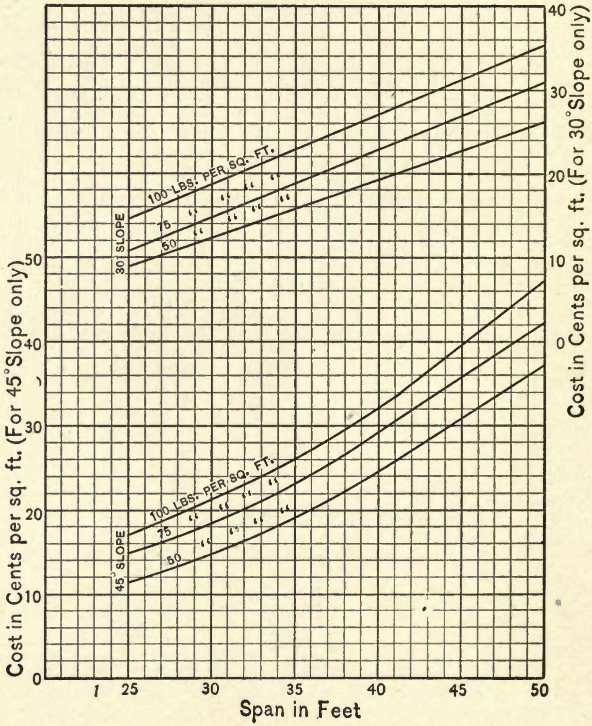
Description of Plots.

In general, each set of plots has been drawn up to render a ready means of determining the cost of the type of truss skeleton per square foot of projected area covered by the different bays and spans. These values should agree very approximately with general straightforward cases, and they are sufficiently accurate for estimating completed work. In special cases they should be modified to suit the case at hand.

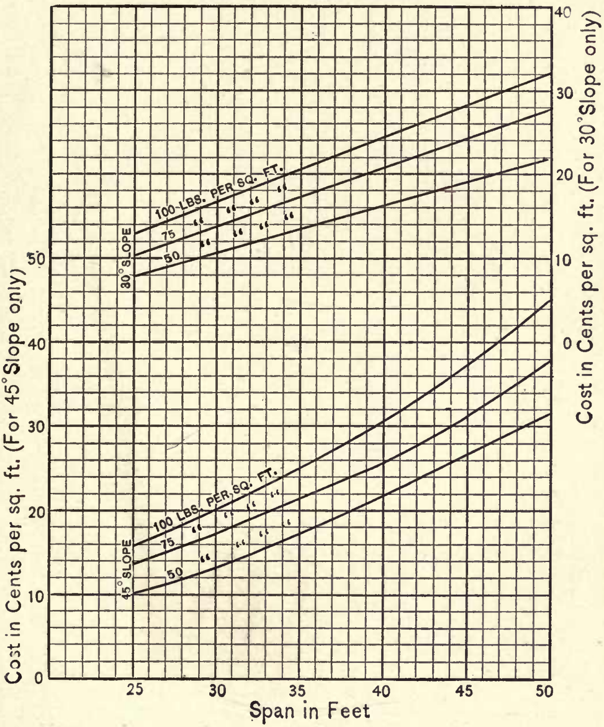
The plots serve a ready means of comparing different types of truss skeleton from a financial standpoint. To bring out this point two diagrams have been plotted: one comparing the types shown under Tables XVI and XVIII; the other, those shown under Tables XVII and XVIII. It may be seem that, for like conditions, the type shown under Table XVIII is more economical than either of the other two, and again, that type "XVI" is cheaper than "XVII."

The cost per square foot of projected area of type XVII*c* is about 10 per cent greater than that of types XVII to XVII*b*, while that of XVII*d* is about 20 per cent greater than that of XVII to XVII*b* for like bays and corresponding spans.

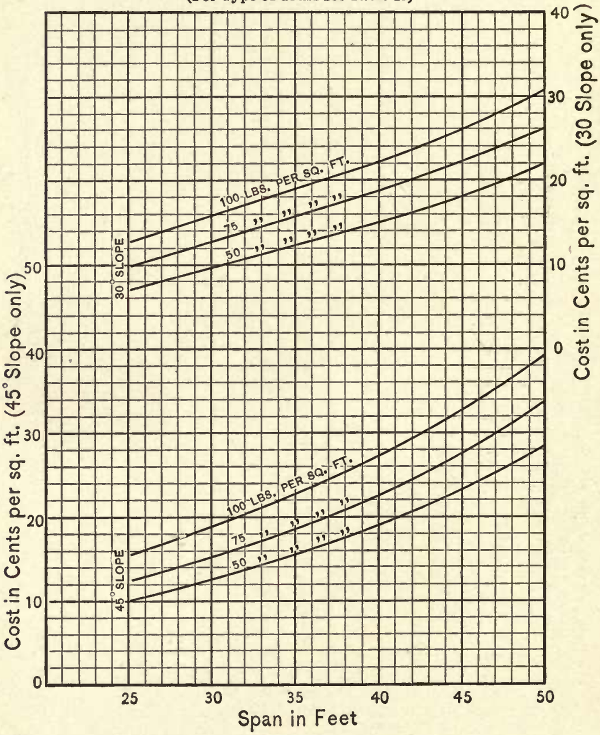
8 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN
(For Truss see Table 15)



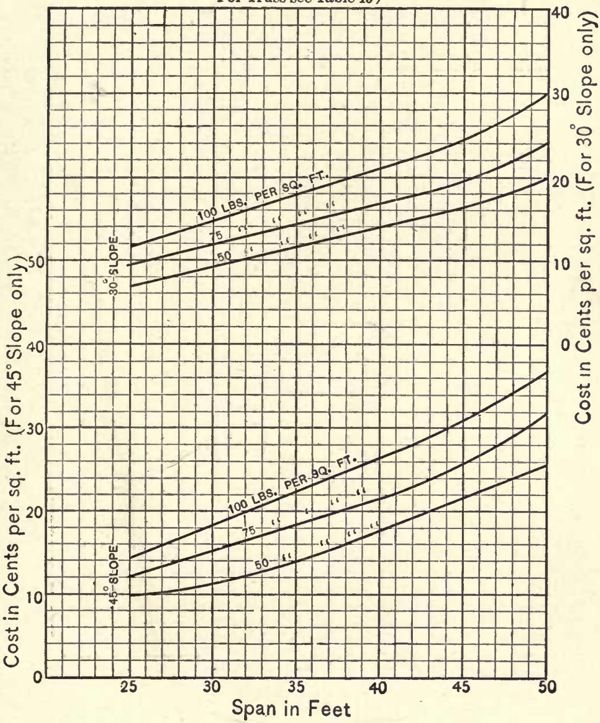
10 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN
(For Type of Truss see Table 15)



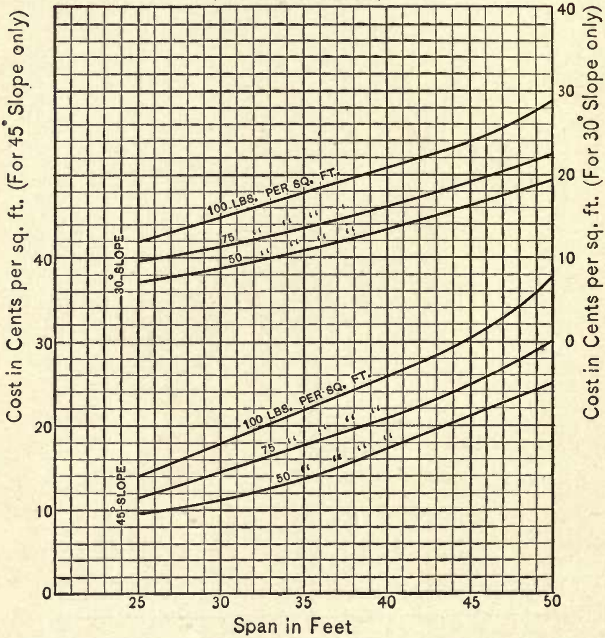
12 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN
(For Type of Truss see Table 15)



14 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN,
For Truss see Table 15)

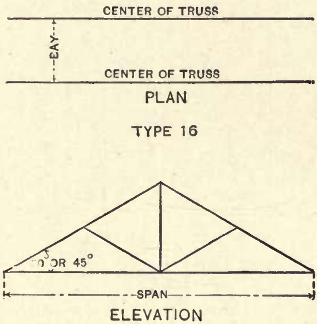


16 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN.
(For Truss see Table 15)



Description of Table XVI.

This table treats the type of truss shown as does Table XV its type. By reference to the description of the latter, this table will be readily understood. No further mention need be made except in cases where excessively long spans cause



diagonal braces longer than thirty times the least dimension of the brace section, in which cases, when the reinforcement will not carry the tension caused by the eccentric loading, either the reinforcement should be increased, or the unsupported length diminished by struts or braces of small section.

Note. — For reinforcement in upper chord, use that required in Table XV for one-half the span given here.

Note. — The reinforcement sizes of lower chord are the same for both the 45° slope and the 30° slope.

Table XVI.

40-Foot Span.

1	2	3	4	5	6	7
	Sizes of Upper Chord.					
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	4×16	5×18	5×20	4×14	5×16	5×18
	5×16	6×18	6.5×20	5×14	6.5×16	7×18
	5×16	5×18	6×20	4×16	5×16	5×20
10	6×16	6.5×18	7.5×20	5.5×16	7×16	7×20
	5×18	5×20	6×22	5×16	5×18	5×20
12	6×18	6.5×20	7.5×22	6.5×16	7×18	7.5×20
	5×18	6×20	6×22	5×16	5×20	6×20
14	6×18	7.5×20	8×22	7×16	7×20	9×20
	5×20	6×22	6×24	5×18	5×20	6×22
16	6×20	7.5×22	8×24	7×18	7.5×20	9×22
	Sizes of Lower Chord.					
	8	9	10	11	12	13
8	4×16	4×16	4×16	....	....	....
	5×16	5×16	5.5×16	5.5×16	6×16	6.5×16
	4×16	4×16	5×16	....	....	....
10	5×16	5.5×16	7×16	5.5×16	6.5×16	8×18
	4×16	4×16	5×18	....	....	....
12	5×16	5.5×16	7×18	6×16	7×16	8×16
	4×16	5×16	5×20	....	....	....
14	5×16	7×16	7×20	6×16	7×16	8.5×20
	4×16	5×18	6×20	....	....	....
16	5.5×16	7×18	8×20	6.5×16	7.5×16	10×20

	Area of Truss Rod (sq. in.).			Diagonal Braces.					
1	14	15	16	17	18	19	20	21	22
Bay (feet).	Load sq. ft. (lbs.).			45° Slope. Load sq. ft. (lbs.).			30 Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100	50	75	100
8.....	1.38	1.98	2.58	4×3	4×5	4×6	4×4	4×6	4×7
10.....	1.68	2.43	3.22	4×4	4×6	5×6	4×5	4×7	5×7
12.....	1.78	2.88	3.88	4×5	4×7	5×7	4×5	4×8	5×9
14.....	2.28	3.37	4.48	4×5	5×6	5×8	4×6	5×8	5×10
16.....	2.58	3.88	5.13	4×6	5×7	6×8	4×7	5×9	6×10

TABLE XVI. — Continued.

50-Foot Span.

1	Sizes of Upper Chord. 2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	6×18	7×20	7.5×22	6.5×16	7×18	8×20
10	6×20	7.5×22	7.5×24	6.5×18	8×20	8.5×22
12	7×20	7.5×24	9×26	7×18	8.5×20	8.5×24
14	7.5×22	8.5×26	9×26	7×20	8.5×22	9×24
16	7.5×22	9×26	9×28	8×20	8.5×24	10.5×24

	Sizes of Lower Chord. 8	9	10	11	12	13
	5×20	5×20	5×20	....	....	....
8	6×20	6×20	6.5×20	6.5×20	7×20	7.5×20
	5×20	5×20	5×20	....	....	....
10	6×20	6.5×20	7.5×20	6.5×20	7.5×20	8.0×20
	5×20	5×20	6×20	....	....	....
12	6×20	6.5×20	8×20	7×20	7.5×20	9.5×20
	5×20	5×20	6×22	....	....	....
14	6.5×20	7×20	8×22	7.5×20	8×20	10×22
	5×20	6×20	6×24	....	....	....
16	6.5×20	8×20	8.5×24	7.5×20	9.5×20	10×24

	Area of Truss Rod (sq. in.).			Diagonal Braces.					
1	14	15	16	17	18	19	20	21	22
Bay (feet).	Load sq. ft. (lbs.)			45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100	50	75	100
8.....	1.8	2.6	3.4	5×3	5×5	5×6	5×4	5×6	5×7
10.....	2.2	3.2	4.0	5×4	5×6	5×7	5×5	5×7	5×9
12.....	2.6	3.8	5.0	5×5	5×7	6×7	5×6	5×8	6×9
14.....	3.0	4.4	5.8	5×5	5×8	6×9	5×6	5×9	6×10
16.....	3.4	4.9	6.6	5×6	6×7	6×10	5×7	6×9	6×12

TABLE XVI. — Continued.

60-Foot Span.

1	2	3	4	5	6	7
	Sizes of Upper Chord.					
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	7×20	7×24	8×26	6.5×20	8×20	8×24
10	7×22	8.5×24	8.5×28	7.5×20	8×22	9.5×24
12	7.5×24	8.5×26	10×28	8×20	8.5×22	10×26
14	7.5×24	8.5×28	10×30	8×22	9.5×24	10×28
16	7.5×26	10×28	11.5×32	8×24	10×26	11.5×28

	Sizes of Lower Chord.					
	8	9	10	11	12	13
	6×24	6×24	6×24	....	....	....
8	7×24	7×24	7.5×24	7.5×24	8×24	8.5×24
	6×24	6×24	6×24	....	....	....
10	7×24	7.5×24	8×24	7.5×24	8.5×24	9×24
	6×24	6×24	6×24	....	....	....
12	7×24	7.5×24	8×24	8×24	8.5×24	9.5×24
	6×24	6×24	7×24	....	....	....
14	7.5×24	8×24	9.5×24	8×24	9×24	11×24
	6×24	6×24	7×24	....	....	....
16	7.5×24	8×24	10×24	8.5×24	9.5×24	11.5×24

	Area of Truss Rod (sq. in.).			Diagonal Braces.					
1	14	15	16	17	18	19	20	21	22
Bay (feet).	Load sq. ft. (lbs.).			45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100	50	75	100
8.....	2.4	3.3	4.2	6×3	6×4	6×6	6×4	6×5	6×7
10.....	2.9	4.0	5.1	6×4	6×5	6×7	6×5	6×7	6×9
12.....	3.3	4.7	6.0	6×4	6×6	6×8	6×5	6×8	6×10
14.....	3.8	5.3	7.0	6×5	6×7	7×8	6×6	6×9	7×10
16.....	4.2	6.0	7.9	6×6	6×8	7×9	6×7	6×10	7×12

TABLE XVI. — Continued.

70-Foot Span.

1	Sizes of Upper Chord.					
	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	7×24	8×28	9.5×28	7.5×20	8×24	9×26
10	8×26	9.5×28	9.5×32	7.5×22	9×26	9.5×28
12	8.5×28	10×30	11×32	8×24	9.5×26	11×28
14	8.5×28	10×32	11×34	9×24	10.5×28	11×32
16	9.5×28	11×32	11.5×36	9.5×26	11×30	11.5×32

	Sizes of Lower Chord.					
	8	9	10	11	12	13
	Reinforcement Size, 7 × 28 for whole span.					
8	8×28	8×28	8.5×28	8.5×28	9×28	9.5×28
10	8×28	8.5×28	9×28	8.5×28	9.5×28	10×28
12	8×28	8.5×28	9×28	9×28	10×28	10.5×28
14	8.5×28	9×28	9.5×28	9×28	10×28	11×28
16	8.5×28	9×28	10×28	9.5×28	10.5×28	11.5×28

	Area of Truss Rod (sq. in.).			Diagonal Braces.					
1	14	15	16	17	18	19	20	21	22
Bay (feet).	Load sq. ft. (lbs.).			45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100	50	75	100
8.....	3.05	4.10	5.15	7×3	7×4	7×5	7×3	7×5	7×7
10.....	3.58	4.89	6.20	7×3	7×5	7×6	7×4	7×6	7×8
12.....	4.10	5.68	7.25	7×4	7×6	7×8	7×5	7×7	7×10
14.....	4.58	6.59	8.30	7×5	7×7	7×9	7×6	7×8	7×11
16.....	5.15	7.25	9.35	7×5	7×8	7×10	7×7	7×10	7×13

TABLE XVI. — Continued.

80-Foot Span.

1	2	3	4	5	6	7
	Sizes of Upper Chord.					
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	8×28	9.5×30	9.5×32	7.5×24	9×26	10.5×28
10	9×28	9.5×32	11×34	9×24	10×28	11.5×32
12	9.5×30	10.5×32	11×36	10×28	11.5×32	12×36
14	9.5×32	11×34	12×38	10×30	11.5×34	13.5×36
16	10.5×32	11×36	12.5×40	11×32	13×36	13.5×40

1	8	9	10	11	12	13
	Sizes of Lower Chord.					
	Reinforcement Size, 8×32 for whole span.					
8	9×32	9×32	9.5×32	9.5×32	10×32	10.5×32
10	9×32	9.5×32	10×32	9.5×32	10.5×32	11×32
12	9×32	9.5×32	10.5×32	10×32	10.5×32	11.5×32
14	9.5×32	10×32	10.5×32	10×32	11×32	12×32
16	9.5×32	10×32	11×32	10.5×32	11.5×32	13×32

1	14	15	16	17	18	19	20	21	22
	Area of Truss Rod (sq. in.).			Diagonal Braces.					
Bay (feet).	Load sq. ft. (lbs.)			45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100	50	75	100
8.....	3.8	5.0	6.2	8×3	8×4	8×5	8×3	8×5	8×6
10.....	4.4	5.9	7.4	8×3	8×5	8×6	8×4	8×6	8×8
12.....	5.0	6.8	8.6	8×4	8×6	8×7	8×5	8×7	8×9
14. ...	5.6	7.7	9.8	8×4	8×6	8×8	8×5	8×8	8×11
16.....	6.2	8.6	11.0	8×5	8×7	8×10	8×6	8×9	8×12

TABLE XVI. — Continued.

90-Foot Span.

1	2	3	4	5	6	7
	Sizes of Upper Chord.					
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	9×30	10.5×32	10.5×36	8.5×26	10×28	10.5×24
10	9×32	10.5×36	12×38	×28	10×32	11.5×33
12	10.5×32	11.5×36	12×40	10×28	11.5×32	12×36
14	10.5×34	12×38	13×42	10×30	12×34	13.5×36
16	12×36	12×40	13.5×44	11.5×32	12×36	13.5×40

	8	9	10	11	12	13
	Sizes of Lower Chord.					
	Reinforcement Size, 9×36 for whole span.					
8	10×36	10×36	10.5×36	10.5×36	11×36	11.5×36
10	10×36	10.5×36	11×36	10.5×36	11.5×36	11.5×36
12	10×36	10.5×36	11×36	11×36	12×36	12.5×36
14	10.5×36	11×36	11.5×36	11×36	12×36	13×36
16	10.5×36	11×36	12×36	11.5×36	12.5×36	13.5×36

1	14	15	16	17	18	19	20	21	22
	Area of Truss Rod (sq. in.).			Diagonal Braces.					
Bay (feet).	Load sq. ft. (lbs.).			45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	30	75	100	50	75	100	50	75	100
8.....	4.7	6.1	7.4	9×3	9×4	9×5	9×3	9×5	9×6
10.....	5.4	7.1	8.8	9×3	9×5	9×6	9×4	9×6	9×7
12.....	6.1	8.2	10.2	9×4	9×5	9×7	9×5	9×7	9×9
14.....	6.8	9.2	11.6	9×4	9×6	9×8	9×5	9×8	9×10
16.....	7.4	10.1	12.8	9×5	9×7	9×9	9×6	9×9	9×12

TABLE XVI. — Continued.

100-Foot Span.

1	Sizes of Upper Chord.					
	2	3	4	5	6	7
Bay (feet).	45° slope. Load sq. ft. (lbs.).			30° slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	10×34	11.5×36	11.5×40	9.5×28	10×32	11.5×34
10	11×36	11.5×40	12.5×42	10.5×30	11.5×34	13×36
12	11.5×36	12.5×40	13×44	11×32	11.5×36	13×40
14	11.5×38	13×42	14.5×44	11×34	13×38	14.5×42
16	11.5×40	13×44	15.5×48	12.5×36	13×40	15×44
	Sizes of Lower Chord.					
	8	9	10	11	12	13
	Reinforcement Size, 10×38 for whole span.					
8	11×38	11×38	11.5×38	11.5×38	12×38	12.5×38
10	11×38	11.5×38	12×38	11.5×38	12.5×38	13×38
12	11×38	11.5×38	12×38	12×38	13×38	13.5×38
14	11.5×38	12×38	12.5×38	12.5×38	13.5×38	14×38
16	11.5×38	12.5×38	13×38	13×38	14×38	15×38

Area of Truss Rod (sq. in.).				Diagonal Braces.					
1	14	15	16	17	18	19	20	21	22
Bay (feet).	Load sq. ft. (lbs.).			45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100	50	75	100
8.....	5.6	7.1	8.6	10×3	10×4	10×5	10×3	10×4	10×6
10.....	6.4	8.2	10.1	10×3	10×4	10×6	10×4	10×5	10×7
12.....	7.1	9.4	11.6	10×4	10×5	10×7	10×4	10×6	10×8
14.....	7.9	10.2	13.1	10×4	10×6	10×8	10×5	10×7	10×10
16.....	8.6	11.6	14.6	10×5	10×7	10×9	10×6	10×8	10×11

Description of Table XVIa.

Like Table XV*a*, this table has computed values of the weights of various truss skeletons, of the type shown under Table XVI, per square foot of projected area for different bays and spans. Like uses may be made of this data, as stated under the description of Table XV*a*.

Table XVI*a*. — *Weight of Truss Skeleton per Square Foot of Area Covered.*

40-Foot Span.

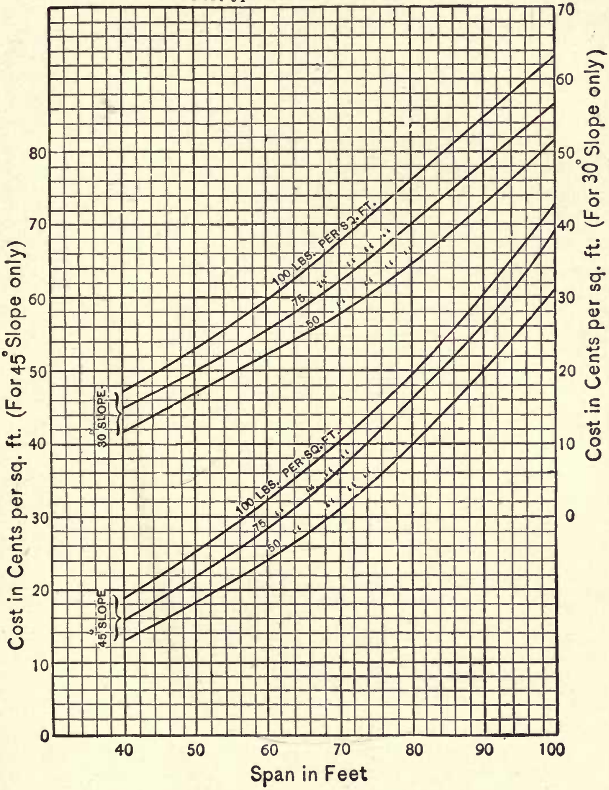
1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	26	32	38	23	30	34
10	23	28	36	21	26	32
12	21	26	33	20	24	28
14	19	26	31	18	22	30
16	18	25	30	17	21	30
Average	21	27	34	18	25	31
50-Foot Span.						
8	37	44	50	34	39	47
10	32	40	45	35	37	42
12	29	36	45	26	32	40
14	28	36	41	24	30	37
16	25	34	40	23	29	37
Average	30	38	44	28	33	41
60-Foot Span.						
8	49	55	65	45	52	59
10	42	51	59	39	53	52
12	38	44	53	34	38	48
14	33	41	51	31	37	46
16	30	40	48	29	36	44
Average	38	46	55	36	43	50

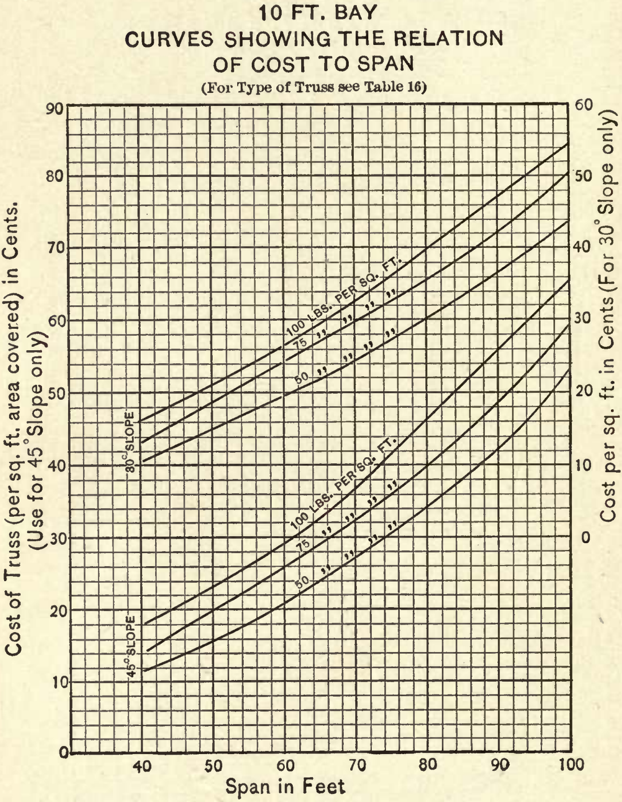
TABLE XVIa. — Continued.

70-Foot Span.

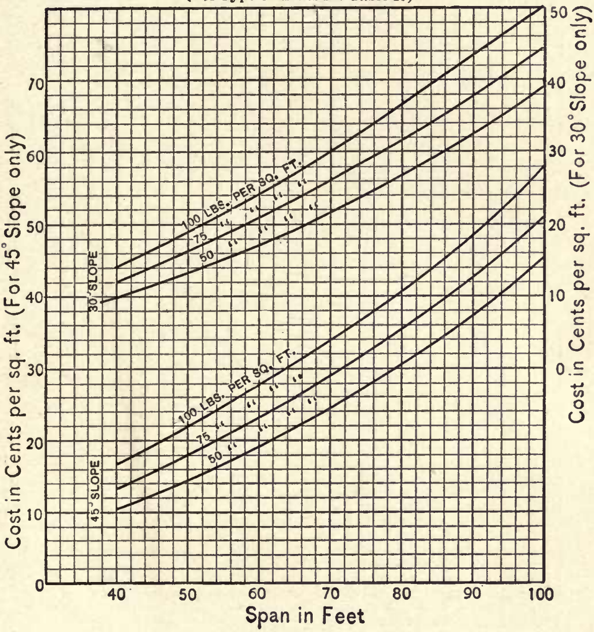
1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
8	62	73	85	56	64	75
10	56	68	74	48	60	63
12	51	60	69	43	52	60
14	45	55	64	39	49	57
16	43	52	60	39	47	52
Average	51	62	70	45	54	61
80-Foot Span.						
8	81	93	99	69	80	92
10	69	79	92	60	71	84
12	62	71	81	58	69	79
14	56	66	76	52	62	74
16	53	60	73	50	62	71
Average	64	74	84	58	69	80
90-Foot Span.						
8	100	113	122	87	98	109
10	83	98	113	73	85	95
12	75	87	97	65	78	87
14	75	81	92	58	71	81
16	67	73	87	57	65	76
Average	80	90	102	68	79	90
100-Foot Span.						
8	122	136	146	103	113	127
10	106	118	131	87	101	108
12	91	100	116	78	89	101
14	83	96	108	71	85	97
16	74	88	106	69	78	91
Average	95	108	121	82	93	105

8 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN,
For Type of Truss see Table 16

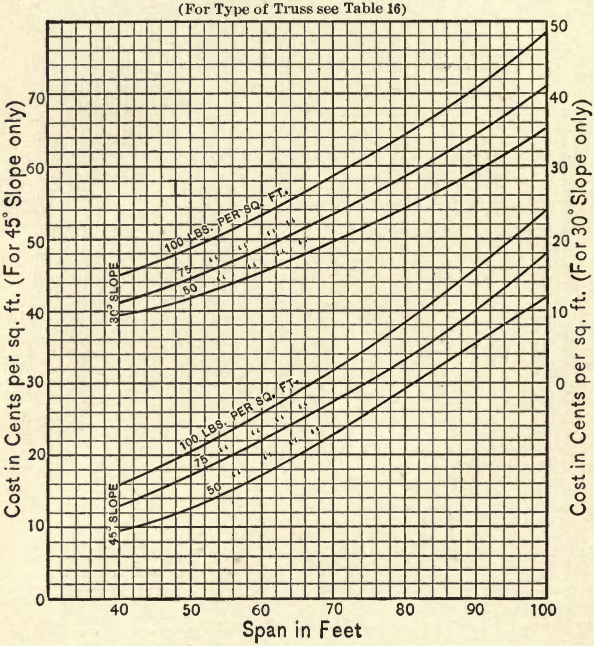



10 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN
(For Type of Truss see Table 16)
Cost of Truss (per sq. ft. area covered) in Cents.
(Use for 45° Slope only)
Cost per sq. ft. in Cents (For 30° Slope only)
30° SLOPE
45° SLOPE
100 LBS. PER SQ. FT.
75 " " "
50 " " "
90
80
70
60
50
40
30
20
10
0
60
50
40
30
20
10
0
40
50
60
70
80
90
100
Span in Feet

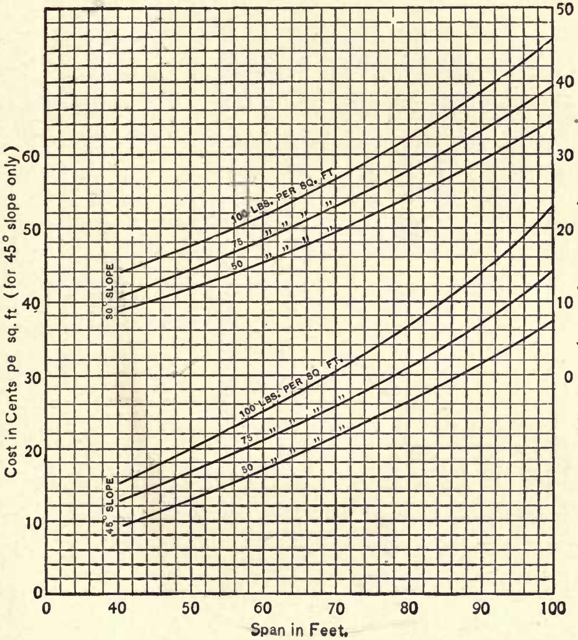
12 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN.
(For Type of Truss see Table 16)



14 FT. BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN.
(For Type of Truss see Table 16)

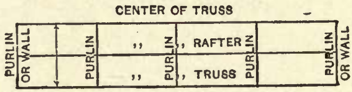


16 FOOT BAY
CURVES SHOWING THE RELATION
OF COST TO SPAN
For Type of truss see Table 16



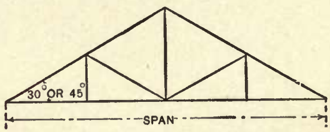
Description of Table XVII.

The type of truss here shown is treated similarly to those shown under Tables XV and XVI, giving complete designs of all the members. The only peculiarity of this table over Table XVI is that, to reduce the roof span on account of widening the bays, intermediate rafters carried on purlins, have been interposed as the sketch will clearly show. It has been figured to keep the same size for the purlins as for the intermediates.



PLAN

TYPE 17



ELEVATION

To do so the span of the purlin can be but one-half that of the intermediates for a given case, since there is a concentrated load on the former equal to the uniformly distributed load on the latter. To reduce the span of the purlins the amount just stated, purlin braces have been figured to be placed diagonally between the underside of the purlins and their adjacent vertical tie members at the panel points. These braces serve as sway bracing to the lower chords of the main trusses as well.

TABLE XVII. — *Sizes of Intermediates. Sizes of Purlins.*

30-Foot Span.

Bay (feet).	Upper Chord. Reinforcement Sizes. 45° Slope. Load per sq. ft. (lbs.).			30° Slope. Load per sq. ft. (lbs.).		
	50	75	100	50	75	100
18	4×14	4×16	5×18	3×14	4×16	5×16
20	4×16	5×16	5×18	3×14	4×16	5×16

40-Foot Span.

18	5×18	5×20	6×22	4×16	5×16	5×20
20	5×18	6×20	6×22	4×16	5×18	5×20

50-Foot Span.

18	6×20	6×24	7×26	5×16	5×20	6×22
20	6×22	7×24	7×28	5×18	6×20	6×22

60-Foot Span.

18	6×24	7×28	8×30	6×20	6×20	7×24
20	7×24	7×28	8×30	6×20	6×24	7×26

70-Foot Span.

18	7×28	8×30	9×34	6×24	7×26	8×28
20	8×28	8×32	9×34	6×24	7×26	8×28

30-Foot Span.

Bay (feet).	Reinforcement Sizes, any slope.	Lower Chord. Concrete Sizes. 45° Slope. Load per sq. ft. (lbs.).			30° Slope. Load per sq. ft. (lbs).		
		50	75	100	50	75	100
18......	2.5×12	4×12	5×12	6×12	6×12	7×12	9×12
20......	2.5×12	5×12	6×12	7×12	7×12	8×12	9×12

40-Foot Span.

18......	4×16	6×16	7×16	8×16	7×16	9×16	10×16
20......	4×16	6×16	7×16	8×16	8×16	9×16	11×16

TABLE XVII. — Continued.

50-Foot Span.

Bay (feet).	Reinforcement Sizes, any slope.	Lower Chord. Concrete Sizes. 45° Slope. Load per sq. ft. (lbs.). 50	75	100	30° Slope. Load per sq. ft. (lbs.). 50	75	100
18.....	6 × 20	8 × 20	9 × 20	10 × 20	9 × 20	10 × 20	12 × 20
20......	6 × 20	8 × 20	9 × 20	10 × 20	10 × 20	11 × 20	13 × 20
60-Foot Span.							
18......	6 × 24	8 × 24	9 × 24	10 × 24	9 × 24	11 × 24	12 × 24
20......	6 × 24	8 × 24	9 × 24	10 × 24	10 × 24	11 × 24	13 × 24
70-Foot Span.							
18......	7 × 28	9 × 28	10 × 28	11 × 28	10 × 28	12 × 28	13 × 28
20......	7 × 28	9 × 28	10 × 28	11 × 28	11 × 28	12 × 28	11 × 28

30-Foot Span.

Bay (feet).	Upper Chord. Concrete Sizes. 45° Slope. Load per sq. ft. (lbs.). 50	75	100	30° Slope. Load per sq. ft. (lbs.). 50	75	100
18	6 × 14	7 × 16	8 × 18	6 × 14	8 × 16	10 × 16
20	6 × 16	8 × 16	9 × 18	6 × 14	8 × 16	10 5 × 16
40-Foot Span.						
18	7 × 18	8 × 20	10 × 22	7.5 × 16	10 × 16	10.5 × 20
20	8 × 18	9 × 20	10 × 22	8 × 16	10 × 18	11 × 20
50-Foot Span.						
18	9 × 20	9 × 24	11 × 26	9.5 × 16	10 × 20	12.5 × 22
20	9 × 22	10 × 24	12 × 28	9.5 × 18	11.5 × 20	13 × 22
60-Foot Span.						
18	9 × 24	10 × 28	13 × 30	10.5 × 20	12 × 20	14 × 24
20	10 × 24	11 × 28	13 × 30	11 × 20	12 × 24	14 × 26
70-Foot Span.						
18	10 × 28	12 × 30	13 × 34	10 × 24	12.5 × 26	15 × 28
20	12 × 28	12 × 32	13 × 34	10.5 × 24	14 × 26	15.5 × 28

Table XVII. — Continued.

30-Foot Span. Diagonal Braces.

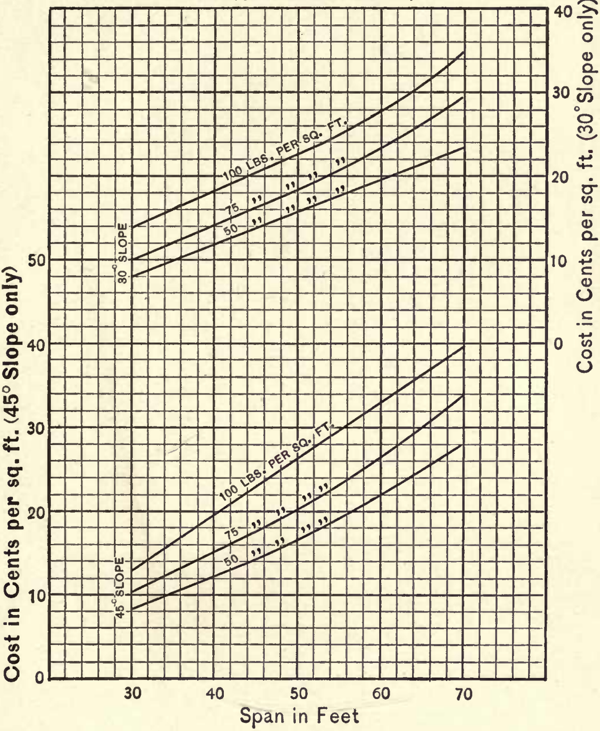
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.)		
	50	75	100	50	75	100
18	4×6	5×7	6×8	6×6	7×7	9×9
20	5×5	6×6	7×7	7×6	8×7	9×8
40-Foot Span.						
18	6×6	7×7	8×8	7×7	9×8	10×10
20	6×6	7×8	8×9	8×7	9×9	11×11
50-Foot Span.						
18	8×7	9×8	10×10	9×10	10×12	12×13
20	8×7	9×9	10×11	10×10	11×12	13×13
60-Foot Span.						
18	8×8	9×10	10×13	9×14	11×15	12×17
20	8×9	9×12	10×14	10×14	11×17	13×17
70-Foot Span.						
18	9×10	10×12	11×14	10×15	12×17	13×20
20	9×12	10×12	11×15	11×16	12×19	14×20
30-Foot Span. Purlin Braces.						
18	3×3	3×4	4×4	3×4	4×5	5×5
20	3×3	3×4	4×4	3×4	4×5	5×5
40-Foot Span.						
18	4×4	5×5	×6	5×5	6×6	6×8
20	4×4	5×5	×6	5×5	6×6	6×8
50-Foot Span.						
18	5×5	6×6	7×7	6×6	7×8	7×10
20	5×5	6×6	7×7	6×6	7×8	7×10
60-Foot Span.						
18	6×6	7×7	8×8	7×7	7×11	8×13
20	6×6	7×7	8×8	7×7	7×11	8×13
70-Foot Span.						
18	7×7	8×9	9×11	8×9	8×13	9×55
20	7×7	8×9	9×11	8×9	8×13	9×15

Table XVII. — Continued.

30-Foot Span. Truss Rods at Apex.

Bay (feet).	45° Slope.			30° Slope.		
	Area in sq. in. Load sq. ft. (lbs.).			Area in sq. in. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
18	.18	1.06	1.43	.93	1.36	1.77
20	.92	1.37	1.74	1.06	1.50	1.94
40-Foot Span.						
18	1.34	1.78	2.36	1.42	2.01	2.60
20	1.37	1.95	2.56	1.53	2.20	2.85
50-Foot Span.						
18	2.04	2.73	3.56	2.44	3.22	4.06
20	2.18	3.00	4.04	2.70	3.66	4.56
60-Foot Span.						
18	2.58	3.59	4.82	3.40	4.40	5.45
20	2.87	3.90	5.20	3.82	4.86	6.00
70-Foot Span.						
18	3.46	4.66	5.95	4.24	5.74	7.19
20	4.02	6.07	6.33	4.94	6.34	7.79
30-Foot Span. Panel Point Rods.						
18	.31	.47	.62	.48	.70	.93
20	.33	.48	.63	.49	.74	.95
40-Foot Span.						
18	.60	.87	1.14	.87	1.28	1.67
20	.60	.87	1.14	.87	1.28	1.67
50-Foot Span.						
18	1.02	1.45	1.88	1.44	2.08	2.73
20	1.02	1.45	1.88	1.44	2.08	2.73
60-Foot Span.						
18	1.40	1.89	2.58	2.03	2.95	3.83
20	1.40	1.89	2.58	2.03	2.95	3.83
70-Foot Span.						
18	2.03	2.87	3.70	2.76	3.99	5.16
20	2.03	2.87	3.70	2.76	3.99	5.16

18-20 FT. BAYS
CURVES SHOWING THE RELATION
OF COST TO SPAN
(For Type of Truss see Table 17)



Description of Table XVIIa.

Under this table may be found values of the weights per square foot of projected area covered of the type of truss skeletons shown in the descriptions of Table XVII for different bays and spans.

Table XVIIa. — *Weight of Truss Skeleton per Square Foot of Area Covered.*

30-Foot Span.

1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
18	16	20	26	15	21	33
20	16	21	26	15	19	31

40-Foot Span.

18	25	30	39	24	27	33
20	24	30	37	24	32	39

50-Foot Span.

18	36	42	52	31	36	45
20	33	42	56	31	38	45

60-Foot Span.

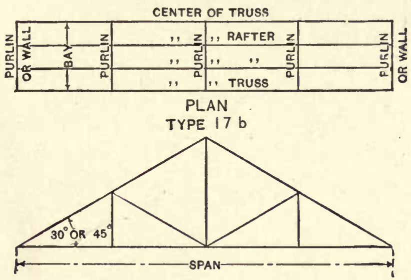
18	42	53	72	39	46	57
20	42	50	67	39	46	54

70-Foot Span.

18	54	67	81	47	60	72
20	59	70	74	46	58	68

DESCRIPTION OF TABLE XVII*b*.

This table, with appended notes, gives complete designs of trusses of the types shown for the spans and bays indicated. The only change from Table XVII is that wider bays are treated. To keep the span of the roof slabs for this type of truss within bounds, two intermediates or rafters are used, carried by purlins, spanning from truss to truss and supported by purlin braces, molded



diagonally between the rafter bearings and the adjacent vertical panel point members. The purlins have been kept the same size as the rafters, to give sufficient bearing for the braces.

Whenever braces are longer than the limit stated under Table XVI, they should be cared for as there stated. This applies to all tables.

TABLE XVII*b*.

30-Foot Span.

Bay (feet).	Upper Chord. Concrete Sizes. 45° Slope. Load sq. ft. (lbs.). 50	75	100	30° Slope. Load sq. ft. (lbs.). 50	75	100
24	7×14	8×16	9×18	7×14	9.5×16	11.5×16
30	7×16	9.5×16	11×18	7.5×14	10×16	13.5×16

40-Foot Span.

24	8×18	9×20	11.5×22	9×16	12×16	12.5×20
30	9.5×18	10.5×20	12×22	10×16	12.5×18	14×20

50-Foot Span.

24	10×20	10×24	12.5×26	11×16	12×20	13.5×22
30	10.5×22	11.5×24	15×28	12×18	14.5×20	15.5×22

60-Foot Span.

24	10×24	11×28	15×30	12×20	14×20	16.5×24
30	11.5×24	13×28	15.5×30	13.5×20	15×24	17.5×26

70-Foot Span.

24	11×28	13.5×30	14.5×34	11.5×24	14.5×26	17.5×28
30	14×28	14×32	15×34	13×24	17.5×26	19×28

Lower Chord. Concrete Sizes.

30-Foot Span.

24	4.5×12	6×12	7.5×12	7.5×12	8.5×12	11×12
30	6.5×12	8×12	9.5×12	9.5×12	11×12	12×12

40-Foot Span.

24	7×16	8×16	9.5×16	8×16	11×16	12×16
30	7×16	8.5×16	10×16	10×16	11.5×16	14.5×16

Table XVII*b*. — Continued.

50-Foot Span.

Bay (feet).	Lower Chord. Concrete Sizes. 45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
24	9 × 20	10 × 20	11.5 × 20	10 × 20	11.5 × 20	14 × 20
30	9 × 20	10.5 × 20	12 × 20	12 × 20	13.5 × 20	16.5 × 20

60-Foot Span.

24	9 × 24	10 × 24	11.5 × 24	10 × 24	13 × 24	14 × 24
30	9 × 24	10.5 × 24	12 × 24	12 × 24	13.5 × 24	16.5 × 24

70-Foot Span.

24	10 × 28	11 × 28	12.5 × 28	11 × 28	14 × 28	15 × 28
30	10 × 28	11.5 × 28	13 × 28	13 × 28	14.5 × 28	17.5 × 28

30-Foot Span. Diagonal Braces.

24	4.5 × 7	6 × 8	7.5 × 8	6 × 8	8 × 9	10 × 10
30	6.5 × 6	8 × 7	9 × 8	8 × 8	9 × 10	10 × 11

40-Foot Span.

24	7 × 7	8 × 8	9.5 × 9	8 × 9	10 × 10	11 × 12
30	7 × 8	8.5 × 9	10 × 11	9 × 10	11 × 11	13 × 13

50-Foot Span.

24	9 × 8	10 × 10	11 × 12	10 × 12	11.5 × 13	14 × 14
30	9 × 9	10 × 12	12 × 12	12 × 12	13.5 × 16	16 × 16

60-Foot Span.

24	9 × 10	10 × 13	11.5 × 14	10 × 16	13 × 17	14 × 19
30	9 × 12	10 × 15	12 × 16	12 × 17	13.5 × 20	16.5 × 20

70-Foot Span.

24	10 × 11	11 × 14	12.5 × 16	11 × 18	14 × 20	15 × 24
30	10 × 15	11.5 × 16	13 × 19	13 × 21	14.5 × 24	17.5 × 24

TABLE XVII*b*. — Continued.

30-Foot Span. Purlin Braces.

Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
24	3×4	4×4	5×5	4×4	5×6	6×6
30	4×4	4×5	5×5	4×5	5×6	6×7

40-Foot Span.

24	4×5	5×6	6×7	5×6	6×8	8×8
30	5×5	6×6	6×8	6×6	7×8	8×9

50-Foot Span.

24	5×6	7×7	8×8	7×7	8×9	10×10
30	6×6	7×8	8×9	7×8	9×9	10×11

60-Foot Span.

24	6×7	8×8	9×10	8×8	10×10	11×12
30	7×7	8×9	10×10	8×9	11×11	12×12

70-Foot Span.

24	8×8	10×10	10×12	9×10	11×12	13×14
30	9×9	10×11	11×12	10×10	12×13	14×14

NOTE. — For reinforcement sizes of upper and lower chords; for sizes of purlins and intermediates or rafters, see Table XVII, noting that the 24 and 30-foot bays here correspond with the 18 and 20-foot bays there, respectively. For truss rod and panel point rod, sizes for 24 and 30-foot bays for this table, refer to Table XVII, and increase the values given for 18 and 20-foot bays for like spans, respectively, by one-third in the first case, and one-half in the second.

24-30 FT. BAYS
CURVES SHOWING THE RELATION OF COST TO SPAN
(For Type of Truss see Table 17-2)

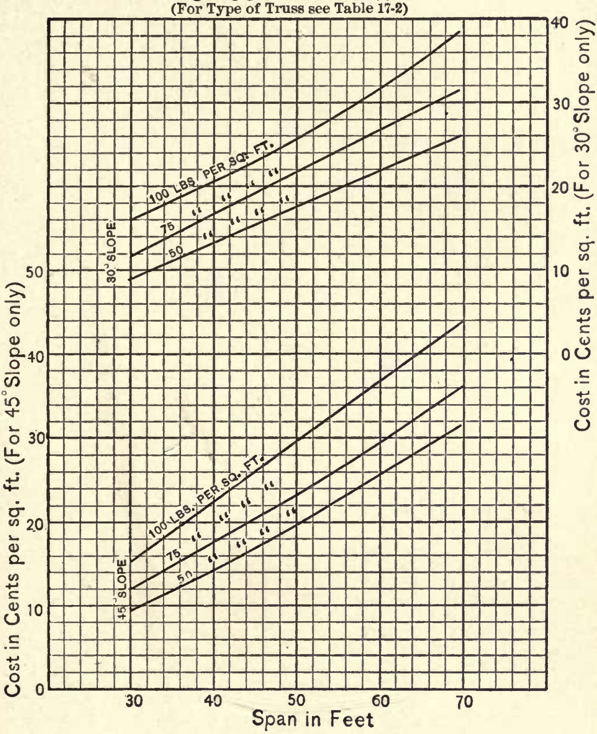


TABLE XVIIb_1. — *Weight of Truss Skeleton per Square Foot of Area Covered*

30-Foot Span

1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
24	19	23	31	18	25	38
30	18	24	30	17	22	33

40-Foot Span.

24	29	36	45	28	31	37
30	27	34	40	27	35	42

50-Foot Span.

24	41	48	60	35	42	53
30	51	46	62	34	42	49

60-Foot Span.

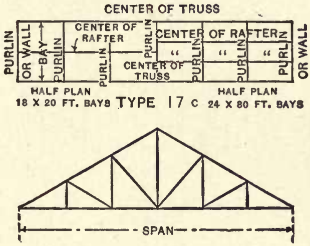
24	48	61	82	45	53	65
30	46	56	74	43	51	60

70-Foot Span.

24	62	76	93	53	68	82
30	65	60	81	51	59	74

Description of Table XVII*c*.

Table XVII*c*, with notes, treats the design of truss here shown for the same bays as Tables XVII and XVII*b*, using similar construction details as



there stated. The only change has been the adding of two more panel points, thereby obtaining satisfactory designs for longer spans.

Table XVII*c*.

45-Foot Span.

1	2	3	4	5	6	7
	Upper Chord. Concrete Sizes.					
Bay (feet).	45° Slope. Load per sq. ft. (lbs.).			30° Slope. Load per sq. ft. (lbs.).		
	50	75	100	50	75	100
18	7 × 14	8.5 × 16	9.5 × 18	7.5 × 14	10 × 16	12.5 × 16
20	7 × 16	9.5 × 16	11 × 18	7.5 × 14	10 × 16	13 × 16
24	8.5 × 14	9 × 16	11 × 18	9 × 14	12 × 16	14.5 × 16
30	8.5 × 16	11.5 × 16	12.5 × 18	9.5 × 16	12.5 × 16	17.5 × 16

NOTE. — For sizes of purlins, intermediates, purlin braces, diagonal braces, truss and panel point rods, and reinforcement sizes for upper and lower chords, see Table XVII, span 30 feet, and corresponding bays.

TABLE XVIIc. — Continued.

60-Foot Span.

	Upper Chord. Concrete Sizes.					
1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
18	8×18	9.5×20	12×22	9.5×16	12.5×16	13.5×20
20	9.5×18	10.5×20	12×22	10×16	12.5×18	14×20
24	9.5×18	11×20	14.5×22	11.5×16	16.5×16	16.5×20
30	11.5×18	12.5×20	15×22	13×16	16.5×18	18.5×20

NOTE. — Corresponds with Table XVII, 40-foot span, for all sizes not given here.

75-Foot Span.

18	10.5×20	10.5×24	13×26	11.5×16	12.5×20	15.5×22
20	10.5×22	11.5×24	14.5×28	11.5×18	14.5×20	16.5×22
24	12×20	12×24	15.5×26	14×16	15.5×20	17.5×22
30	13×22	13×24	19×28	15.5×18	18×20	20.5×22

NOTE. — Corresponds with 50-foot span, Table XVII, for all sizes not given here.

90-Foot Span.

18	10.5×24	11.5×28	15.5×30	13×20	15×20	17.5×24
20	11.5×24	13×28	15.5×30	13.5×20	15×24	17.5×26
24	12×24	13×28	19.5×30	15×20	18×20	21.5×24
30	14×24	16×28	20×30	18×20	19.5×24	22.5×26

105-Foot Span.

18	11.5×28	14×30	15×34	12×24	16×26	18.5×28
20	14×28	14×32	15×34	13×24	17.5×26	20×28
24	13×28	17×30	18×34	15×24	19×26	23×28
30	17×28	17×32	18×34	16.5×24	22.5×26	26×28

NOTE. — For weight per square foot of projected area covered see Tables XVII and XVII*b*, and add about 10 per cent.

Table XVIIc. — Continued.

45-Foot Span.

Bay (feet).	Lower Chord. Concrete Sizes.					
	8	9	10	11	12	13
	45° Slope. Load per sq. ft. (lbs.).			30° Slope. Load per sq. ft. (lbs.).		
	50	75	100	50	75	100
18	5 × 12	6 × 12	7 × 12	7 × 12	9 × 12	11.5 × 12
20	6 × 12	7 × 12	9 × 12	9 × 12	11 × 12	11.5 × 12
24	7 × 12	8 × 12	10 × 12	10 × 12	11.5 × 12	15 × 12
30	8.5 × 12	11.5 × 12	13 × 12	13 × 12	15 × 12	17 × 12

60-Foot Span.

18	7 × 16	8.5 × 16	10 × 16	8.5 × 16	11.5 × 16	13 × 16
20	7 × 16	8.5 × 16	10 × 16	10 × 16	11.5 × 16	14.5 × 16
24	8.5 × 16	10 × 16	12.5 × 16	10 × 16	14.5 × 16	16 × 16
30	8.5 × 16	10.5 × 16	13 × 16	13 × 16	15.5 × 16	19.5 × 16

75-Foot Span.

18	9 × 20	10.5 × 20	12 × 20	10.5 × 20	12 × 20	15 × 20
20	9 × 20	10.5 × 20	12 × 20	12 × 20	13.5 × 20	16.5 × 20
24	10.5 × 20	12 × 20	14.5 × 20	12 × 20	14.5 × 20	18 × 20
30	10.5 × 20	12.5 × 20	15 × 20	15 × 20	17.5 × 20	21.5 × 20

90-Foot Span.

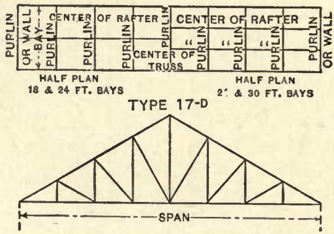
18	9 × 24	10.5 × 24	12 × 24	10.5 × 24	13.5 × 24	15 × 24
20	9 × 24	10.5 × 24	12 × 24	12 × 24	13.5 × 24	16.5 × 24
24	10.5 × 24	12 × 24	14.5 × 24	12 × 24	16.5 × 24	18 × 24
30	10.5 × 24	12.5 × 24	15 × 24	15 × 24	17.5 × 24	21.5 × 24

105-Foot Span.

18	10 × 28	11.5 × 28	13 × 28	11.5 × 28	14.5 × 28	16 × 28
20	10 × 28	11.5 × 28	13 × 28	13 × 28	14.5 × 28	17.5 × 28
24	11.5 × 28	13 × 28	15.5 × 28	13 × 28	17.5 × 28	19 × 28
30	11.5 × 28	13.5 × 28	16 × 28	16 × 28	18.5 × 28	22.5 × 28

Description of Table XVII*d*.

This table, with notes, contains data for designing trusses of the class shown. The only change



over Table XVII*c* is that longer spans have been treated by using eight panels.

Table XVII*d*.

60-Foot Span.

Bay (feet).	Sizes of Upper Chord. 45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
18	8×14	10×16	11×16	9×14	12×16	15×16
20	8×16	11×16	13×18	9×14	12×16	16×16
24	10×14	13×16	13×18	11×14	15×16	18×16
30	10×16	14×16	14×18	12×14	16×16	22×16

80-Foot Span.

18	9×18	11×20	14×22	11×16	15×16	16×20
20	11×18	12×20	14×22	12×16	15×18	17×20
24	12×18	13×20	17×22	14×16	19×16	20×20
30	14×18	13×20	18×22	16×16	20×18	23×20

Table XVIId. — Continued.

100-Foot Span.

Bay (feet).	Sizes of Upper Chord. 45 Slope. Load sq. ft. (lbs.). 50	75	100	30 Slope. Load sq. ft. (lbs.). 50	75	100
18	12×20	12×24	15×26	14×16	15×20	19×22
20	12×22	13×24	17×28	14×18	17×20	20×22
24	14×20	14×24	18×26	18×16	19×20	21×22
30	15×22	16×24	23×28	19×18	23×20	25×22

120-Foot Span.

18	12×24	13×28	18×30	15×20	18×20	14×24
20	13×24	15×28	18×30	16×20	18×24	21×26
24	14×24	15×28	23×30	18×20	22×20	24×24
30	16×24	19×28	23×30	21×20	24×24	28×26

140-Foot Span.

18	13×28	16×30	17×34	14×24	18×26	22×28
20	14×28	16×32	17×34	15×24	21×26	23×28
24	15×28	19×30	20×34	17×24	22×26	25×28
30	20×28	20×32	21×34	20×24	28×26	30×28

60-Foot Span.

Bay (feet).	Sizes of Lower Chord. 45° Slope. Load sq. ft. (lbs.). 50	75	100	30° Slope. Load sq. ft. (lbs.). 50	75	100
18	5.5×12	7.5×12	9.5×12	9.5×12	11.5×12	15.5×12
20	7.5×12	9.5×12	11.5×12	11.5×12	13.5×12	15.5×12
24	6.5×12	9.5×12	12.5×12	12.5×12	14.5×12	19.5×12
30	10.5×12	13.5×12	16.5×12	16.5×12	19.5×12	21.5×12

80-Foot Span.

18	8×16	10×16	12×16	10×16	14×16	16×16
20	8×16	10×16	12×16	12×16	14×16	18×16
24	10×16	12×16	15×16	12×16	18×16	20×16
30	10×16	12×16	16×16	16×16	19×16	25×16

TABLE XVII*d*. — Continued.

100-Foot Span.

Bay (feet).	Sizes of Lower Chord. 45° Slope. Load sq. ft. (lbs.). 50	75	100	30° Slope. Load sq. ft. (lbs.). 50	75	100
18	10×20	12×20	14×20	12×20	14×20	18×20
20	10×20	12×20	14×20	14×20	16×20	20×20
24	12×20	14×20	17×20	14×20	17×20	22×20
30	12×20	15×20	18×20	18×20	21×20	27×20
120-Foot Span.						
18	10×24	12×24	14×24	12×24	16×24	18×24
20	10×24	12×24	14×24	14×24	16×24	20×24
24	12×24	14×24	17×24	14×24	20×24	22×24
30	12×24	15×24	18×24	18×24	21×24	27×24
140-Foot Span.						
18	11×28	13×28	15×28	13×28	17×28	19×28
20	11×28	13×28	15×28	15×28	17×28	21×28
24	13×28	15×28	18×28	15×28	21×28	23×28
30	13×28	16×28	19×28	19×28	22×28	28×28

NOTE. — For all sizes not given here, refer to Tables XVII and XVII*b* to corresponding bays, bearing in mind the following change in spans:

A 60-foot span here corresponds with 30-foot span under Tables XVII and XVII*b*.

A 80-foot span here corresponds with 40-foot span under Tables XVII and XVII*b*.

A 100-foot span here corresponds with 50-foot span under Tables XVII and XVII*b*.

A 120-foot span here corresponds with 60-foot span under Tables XVII and XVII*b*.

A 140-foot span here corresponds with 70-foot span under Tables XVII and XVII*b*.

For weight per square foot of projected area covered, see Tables XVII and XVII*b*, and add about 20 per cent. Reference spans under Tables XVII and XVII*b* are one-half those given here for obtaining corresponding weights.

General Description of Types of Trusses "XVII to XVII*d*."

The upper chord in Tables XVII to XVII*d* have been figured for a uniform section throughout their length, and of a section to withstand the stress at the most stressed part, namely at the center between either of the two bearings and the first panel point from either bearing. In cases where the conditions will allow, it is very economical to vary the section, either by tapering the width uniformly from the bearings to the apex, called "condition A," or by forming steps at each panel point, termed "condition B." The sizes were figured with this in view, as it may be noted that the depths are such as to withstand the bending moment due to the distributed loads for economical widths and, to care for concentrated loads, these widths were increased, keeping the same depths as previously determined. If condition B is adopted, the sizes of sections in the different panels for the tables specified will be as follows:

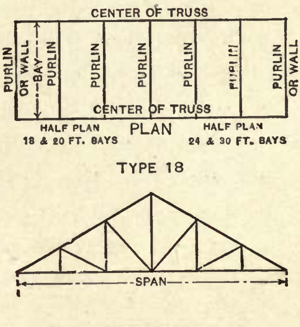
Tables XVII and XVII*b*.

Panel.	Concrete Sizes.
First from apex.....	Reinforcement size + $\frac{3}{4}$ the difference between the reinforcement section and the concrete section given for upper chords. Call this difference x.
Table 17–*c*.	
First from apex.....	Reinforcement size + $\frac{1}{2}x$.
Second from apex...	Reinforcement size + $\frac{5}{6}x$.
Table 17–*d*.	
First from apex.....	Reinforcement size + $\frac{3}{8}x$.
Second from apex...	Reinforcement size + $\frac{5}{8}x$.
Third from apex....	Reinforcement size + $\frac{7}{8}x$.

If condition A is adopted, sizes corresponding with the above should be used at the center of the different panels.

Description of Table XVIII.

This table covers the complete design of a different class of truss, shown by the sketch. In the design the purlins are kept near enough together to carry the roof slabs. Two spacings of purlins have been used, namely 8-foot and 10-foot. The span headings of each set of bays clearly state which of the two panelings to use, or has been used.



The table, as drawn up, has limited the purlin span or the bay to 24 feet. It will be readily seen that this may be increased, using the same sizes of purlins required for the bays given, by molding in braces diagonally between the purlins and the adjacent vertical tie members, so as to

limit the purlin span proper to the values used in the table.

Under "Diagonal Braces" it is noted that the sizes given are for the worst cases. Such cases, as may be seen, occur at the first panel point from the bearings where the braces make angles of 60 degrees when a 30- degree slope of truss is used, and 45 degrees when a 45- degree slope truss is used, with vertical through the panel points in question. Accordingly, the sizes of the other diagonals may be reduced in accordance with the relation that the sines of the angles between verticals through such panel points and their corresponding braces bear to the sine of 60 degrees for a 30-degree slope truss, and to the sine of 45 degrees for a 45-degree slope truss.

Again, it may be discovered that the sizes figured are for 10-foot panels. If 8-foot panels are used instead, the values given may again be reduced 20 per cent. If the first reduction is adhered to, particular attention should be paid to excessively long, unsupported lengths, and the effect of eccentricity thereon as previously treated.

This type of truss may be tapered uniformly, or offset in width over panel points as stated in the general description for Tables XVII to XVII*d*.

General Description of "XVIII" Type Truss.

In designing trusses of this type, the following formula is submitted in determining the total stress in pounds in any part or panel of the upper chord.

Let k = a factor to be determined from the following plot.
b = bay in feet.
s = span in feet.
w = total load per square foot including weight of roof proper.
n = total number of panels into which the span is divided, either 8 or 10 feet.
W = total stress in pounds in any panel.

For 45-degree slope:

1st. Panel from apex, W equals $1.5\,\dfrac{(k.b.s.w)}{n} \times 1.42.$

2d. Panel from apex, W equals $2.5\,\dfrac{(k.b.s.w)}{n} \times 1.42.$

3d. Panel from apex, W equals $3.5\,\dfrac{(k.b.s.w)}{n} \times 1.42.$

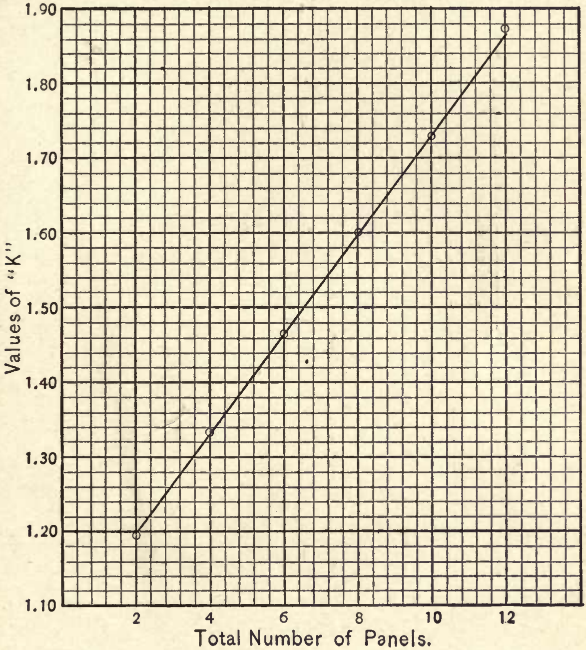
4th. Panel from apex, W equals $4.5\,\dfrac{(k.b.s.w)}{n} \times 1.42.$

etc.

For 30 degree slope use factor 2.0 instead of 1.42 as given for 45 degree slope.

The above stress divided by 500 will give the area of concrete section required for concentrated loading in addition to the section which has to care for the distributed loading and which equals the section so called "Reinforcement Size in Table XVIII."

TABLE 18
PLOT TO DETERMINE VALUES FOR
FACTOR "K" IN FORMULA.



To determine the size of lower chord corresponding to any upper chord to care for concentrated loading, reduce the concrete section of the upper chord for concentrated loading by 29.5 per cent for 45 degree slopes, and by 13.5 per cent for 30 degree slopes. In addition to this section, increase the size by the section required to support this distributed load over a length equal to the panel, as stated earlier in the description.

TABLE XVIII.

30-Foot Span (8-Foot Panels).

1	2	3	4	5	6	7
	Upper Chord. Concrete Sizes.					
	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
Bay (feet).	50	75	100	50	75	100
14	6.5×16	7×20	8.5×20	7×16	7.5×20	9.5×20
16	6.5×18	8.5×20	8.5×24	7×18	9×20	9.5×24
18	7.5×20	8.5×22	10×24	8×20	9×22	10.5×24
20	8×22	9.5×24	10×28	8.5×22	10×24	10.5×28
22	8×24	9.5×26	11×28	8.5×24	10×26	12×28
24	9×24	10.5×28	11×32	9.5×24	11×28	12×32

40-Foot Span (10-Foot Panels).

14	7×18	8.5×20	9×22	7.5×18	9.5×20	10×24
16	7×20	8.5×22	10×24	7.5×20	9.5×22	11×22
18	8×22	9.5×24	10×26	8.5×20	10.5×22	11×26
20	8×24	9.5×28	11×28	8.5×24	11×26	12.5×28
22	9×24	10.5×28	11×32	9.5×24	11.5×28	12.5×32
24	9×28	10.5×30	12×32	9.5×28	11.5×30	13.5×32

TABLE XVIII. — Continued.

50-Foot Span (8-Foot Panels.)

1	2	3	4	5	6	7
	Upper Chord. Concrete Sizes.					
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
14	7.5×16	8×20	10×20	8.5×16	9.5×20	11.5×20
16	7.5×18	9.5×20	10×24	8.5×18	11×20	11.5×24
18	8.5×20	9.5×22	11.5×24	9.5×20	11×22	13×24
20	9×22	10.5×24	11.5×28	10×22	12×24	13×28
22	9×24	11×26	12.5×28	10×24	12×26	14×28
24	10×24	12×28	12.5×32	11×24	13×28	14.5×32

60-Foot Span (10-Foot Panels).

14	8×18	10×20	10.5×32	9×18	11.5×20	12.5×22
16	8×20	11×20	11.5×24	9×20	11.5×22	14×24
18	9×22	11×22	12×26	10×22	13×24	14×26
20	9×24	11×24	13×28	10.5×24	13×26	15×28
22	10.5×24	12.5×26	13×32	11.5×24	14×28	15×32
24	11.5×24	12.5×28	14.5×32	11.5×28	14×30	16.5×32

80-Foot Span (10-Foot Panels).

14	8×18	11.5×20	12.5×22	10.5×18	13.5×20	15×22
16	9×20	11.5×22	14×24	10.5×20	14×22	16×24
18	10×22	12.5×24	14×26	12×22	15×24	17×26
20	10.5×24	13×26	14×28	12×24	15×26	18×28
22	11.5×24	14×28	15×32	13.5×24	16×28	18×32
24	11.5×28	14×30	16.5×32	13×28	16.5×30	19.5×32

100 Foot Span (10 Foot Panels).

14	10×18	13×20	14.5×22	12×18	16×20	17.5×22
16	10×20	13×22	15.5×24	12.5×20	16×22	15×24
18	11.5×22	14.5×24	16×26	14×22	17×24	20×26
20	11.5×24	14.5×26	17×28	14×24	17.5×26	21×28
22	13×24	16×28	17×32	15.5×24	19×28	21×32
24	13×28	16×30	19×32	15.5×28	19×32	23×32

TABLE XVIII. — Continued.

30-Foot Span (8-Foot Panels).

Bay (feet).	Lower Chord. Concrete Sizes.					
	8	9	10	11	12	13
	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
14	4.5×8	5×10	5.5×12	5.5×10	6×12	6.5×14
16	5×8	5×12	6×12	6×10	6.5×12	7.5×14
18	4.5×10	5.5×12	6×12	5.5×12	6×14	8×14
20	5×10	6×12	6.5×12	6×12	6.5×14	8×16
22	5×10	6.5×12	7×12	6.5×12	7×14	8.5×16
24	5×12	6.5×12	7.5×12	7×12	7×16	9×16

40-Foot Span (10-Foot Panels).

14	4.5×10	5.5×12	6×12	5.5×12	6×14	8×16
16	5×10	6×12	6.5×12	6×12	6.5×14	8×16
18	5.5×10	6.5×12	7×12	6.5×12	7×14	8.5×16
20	5×12	6.5×12	7.5×12	7×12	7×16	9×16
22	5.5×12	7×12	7×14	6×14	8×16	9.5×16
24	6×12	7×12	7.5×14	6.5×14	8.5×16	10×16

50-Foot Span (8-Foot Panels).

14	5×12	6×12	7×12	6.5×12	8×14	8.5×16
16	5.5×12	6.5×12	7×14	6×14	8×16	9.5×16
18	6×12	7×12	7.5×14	6.5×14	8.5×16	10×16
20	6.5×12	7.5×12	7.5×16	7×14	9×16	10×18
22	6.5×12	7×14	8×16	7×16	9.5×16	10.5×18
24	7×12	7.5×14	8.5×16	8×16	10×16	10.5×20

TABLE XVIII. — Continued.

60-Foot Span (10-Foot Panels).

Bay (feet).	Upper Chord. Concrete Sizes.					
	8	9	10	11	12	13
	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
14	6.5×12	7×12	7.5×14	6.5×14	8.5×16	10×16
16	6.5×12	7.5×12	7.5×16	7×14	9×16	10×18
18	7×12	7×14	8×16	7×16	9.5×16	10.5×18
20	7×12	7.5×14	8.5×16	8×16	10×16	10.5×20
22	7×12	7.5×16	8.5×18	8.5×16	10×18	10.5×22
24	7.5×12	8×16	9.5×18	9×16	10.5×18	11×24

80-Foot Span (10-Foot Panels).

14	6×14	7.5×14	8×16	8×16	8×20	8×26
16	6.5×14	8×16	8×18	8.5×16	8×22	8.5×28
18	7×14	8.5×16	8×20	8×18	8×24	9×30
20	7.5×14	8×18	8×22	8×20	8×26	10×30
22	7×16	8.5×18	8×24	8.5×20	8×28	11×30
24	7.5×16	9×18	8×26	8×22	8×30	12×30

100-Foot Span (10-Foot Panels).

14	7×14	8.5×16	8×20	8×18	8×24	9×30
16	7.5×16	8×18	8×22	8×20	8×26	10×30
18	8×16	8×20	8×26	8×22	8×30	12×30
20	8.5×16	8×22	8×28	8×24	10×30	13×30
22	8×18	8×24	8.5×28	8×26	11×30	14×30
24	8×18	8×26	9×28	8.5×26	12×30	15×30

Table XVIII. — Continued.

Reinforcement Sizes for Upper Chord.

Bay (feet).	8-Foot Panels.			10-Foot Panels.		
	Load sq. ft. (lbs.)			Load sq. ft. (lbs.).		
	50	75	100	50	75	100
14	5 × 16	5 × 20	6 × 20	5 × 18	6 × 20	6 × 22
16	5 × 18	6 × 20	6 × 24	5 × 20	6 × 22	7 × 24
18	6 × 20	6 × 22	7 × 24	6 × 22	7 × 24	7 × 26
20	6 × 22	7 × 24	7 × 28	6 × 24	7 × 26	8 × 28
22	6 × 24	7 × 26	8 × 28	7 × 24	8 × 28	8 × 32
24	7 × 24	8 × 28	8 × 32	7 × 28	8 × 30	9 × 32

Diagonal Braces (Figured for Worst Case). Sizes in Sq. In.

	45° Slope (10-Foot Panels).			30° Slope (10-Foot Panels).		
14	1	20	26	19	28	38
16	15	23	30	21.5	32	43
18	17	26	34	24	36	48
20	19	29	38	26.5	40	53
22	21	32	42	29.0	44	58
24	23	35	46	31.5	48	63

Truss Rod Sizes at Panel Points. (Sq. In.)

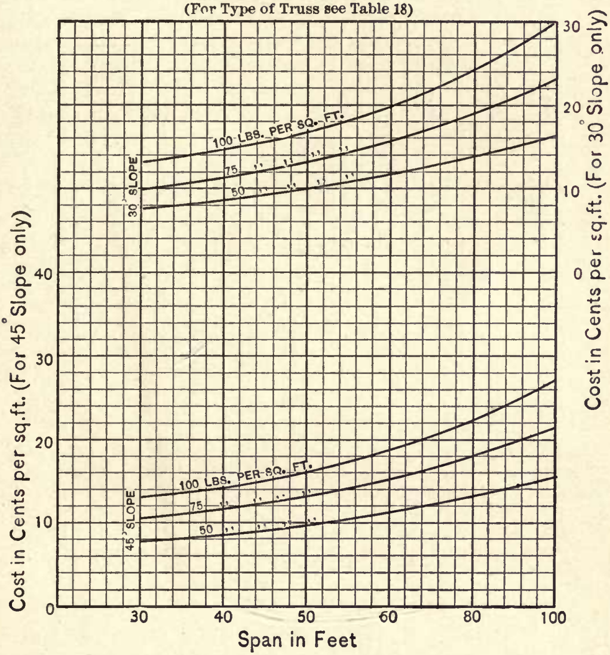
	8-Foot Panels.			10-Foot Panels.		
14	.56	.84	1.12	.70	1.05	1.40
16	.64	.96	1.28	.80	1.20	1.60
18	.72	1.08	1.44	.90	1.35	1.80
20	.80	1.20	1.60	1.00	1.50	2.00
22	.88	1.32	1.76	1.10	1.65	2.20
24	.96	1.44	1.92	1.20	1.80	2.40

For truss rod sizes at apex, double the values given above.

EXPLANATORY NOTE.

For the sizes of purlins and reinforcement for this type of truss, see Table II, Part III, for the corresponding spans and loading.

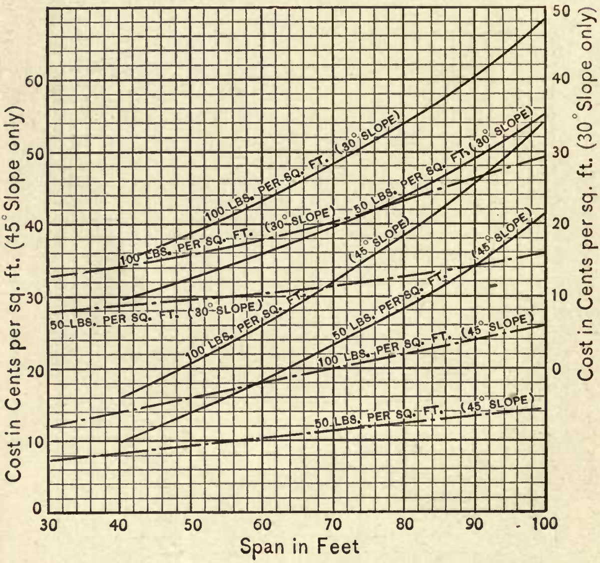
AVERAGE OF BAYS 14-24 FT.
CURVES SHOWING THE RELATION
OF COST TO SPAN
(For Type of Truss see Table 18)



For the reinforcement sizes of the lower chord first determine the total weight between panel points of the concrete sizes here given, and then by reference to Table II, Part III, the sizes of

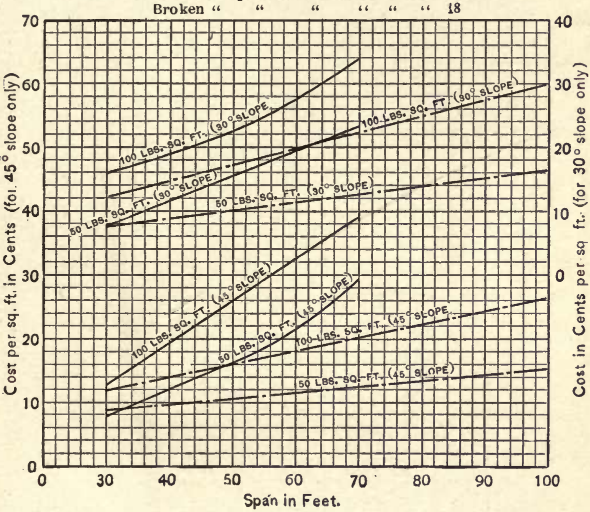
(AVERAGE BAY 14 FT.)
CURVES SHOWING COMPARATIVE COSTS
OF DIFFERENT KINDS OF TRUSSES

Full lines represent truss shown in Table 16
Broken " " " " " " 18



AVERAGE BAY 20 FEET.
CURVES SHOWING COMPARATIVE COSTS
OF DIFFERENT KINDS OF TRUSSES.

Full lines represent truss shown in Table 17
Broken " " " " " " 18



beams to carry these distributed loads for the required spans may be found. Again, by referring this latter size to Table I, Part III, the required reinforcement may be found.

TABLE XVIIIa.—*Weight of Truss Skeleton per sq. ft. of Area Covered.*

30-Foot Span (8-Foot Panels).

1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
14	14	19.5	24	14.3	18.5	24.4
16	14	20.5	25	14.2	19.6	24.5
18	15.5	20.5	25.5	15.2	19.2	25.
20	16.5	21.8	26.3	15.6	20.1	25.7
22	16.5	21.3	26.4	15.5	20.0	26.3
24	17.	22.5	28.5	15.7	21.4	27.2
Average	15.6	20.9	25.9	15.1	19.8	26.6

40-Foot Span (10-Foot Panels).

14	17.6	24.5	28.3	17.7	23.8	28.8
16	17.2	23.4	29.1	16.9	23.0	30.0
18	18.4	24.5	28.2	16.6	22.3	28.6
20	18.3	25.8	27.7	18.5	24.3	30.5
22	18.6	25.1	30.2	17.3	24.9	29.9
24	19.5	24.4	30.2	18.6	24.6	30.5
Average	18.3	24.6	28.9	17.4	23.8	29.7

50-Foot Span (8-Foot Panels).

14	18.3	25.1	29.9	18.2	26.0	31.6
16	18.1	25.2	31.0	18.3	26.3	32.6
18	19.4	25.0	31.3	19.0	25.6	32.0
20	19.9	26.4	32.6	19.3	26.2	33.1
22	19.4	26.7	32.4	19.4	24.9	32.2
24	19.7	27.2	33.4	19.8	26.6	34.3
Average	19.1	26.1	31.7	19.0	25.9	32.6

TABLE XVIIIa. — Continued.

60-Foot Span (10-Foot Panels).

1	2	3	4	5	6	7
Bay (feet).	45° Slope. Load sq. ft. (lbs.).			30° Slope. Load sq. ft. (lbs.).		
	50	75	100	50	75	100
14	22.7	30.2	35.9	22.4	32.2	38.7
16	21.6	28.8	36.7	21.2	30.9	39.5
18	22.8	30.0	36.4	22.9	31.6	37.9
20	22.1	29.3	37.5	23.4	30.7	38.8
22	22.9	32.0	38.8	23.0	31.9	39.8
24	23.6	31.0	39.4	23.8	31.1	40.0
Average	22.6	30.2	37.5	22.8	31.4	39.1

80-Foot Span (10-Foot Panels).

14	24.0	35.0	43.5	27.3	38.0	47.4
16	24.9	35.9	45.3	26.8	37.3	48.0
18	26.1	36.7	44.0	27.8	38.0	48.7
20	26.7	36.1	42.5	27.4	37.0	49.5
22	26.1	37.0	46.0	28.4	38.0	50.7
24	27.4	36.5	46.5	27.6	37.9	50.5
Average	25.9	36.2	44.6	27.6	37.7	49.1

100-Foot Span (10-Foot Panels).

14	29.3	42.5	51.8	31.5	45.0	57.8
16	29.4	40.4	49.5	31.5	43.9	58.0
18	31.8	42.5	52.4	32.8	44.4	60.0
20	30.7	41.6	52.8	32.3	46.2	60.0
22	30.8	43.5	54.0	32.4	49.0	61.0
24	31.6	43.0	54.2	33.4	49.2	61.0
Average	30.6	42.6	54.1	32.3	46.3	59.6

THEORY OF STEEL-CONCRETE ARCHES AND OF VAULTED STRUCTURES

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CONTENTS.